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GEOLOGIC AND GEOPHYSICAL STUDIES RELATED TO CONSTRUCTION OF THE TRANSMOUNTAIN PIPELINE IN CLALLAM COUNTY, WASHINGTON



by

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August 1981

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ABSTRACT

Geologic and Geophysical Studies Related to Construction of the TransMountain Pipeline in Clallam County, Washington.

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This report presents the results of a review and analysis of geologic and geophysical information submitted to Clallam County by the TransMountain Pipeline Company as part of their proposal for construction of a marine pipeline facility and pipeline which passes through the county. This report addresses the seismicity of the area, reviews the groundwater problem, evaluates the Low Point off-loading and tank farm sites, evaluates proposed anchor penetration standards, and analyzes the slope stability and sediment liquefaction potential for the submarine crossings. The primary conclusion that may be drawn from this review is that Trans Mountain Pipeline has not provided sufficient information to adequately assess their construction proposal.

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SECTION I: GROUNDWATER

Introduction

The TransMountain Pipeline Company (TMPC) has applied for site certification to build an oil pipeline through the eastern part of Clallam County, Washington. The pipeline would begin at a dock and tank farm at Low Point, extend east along the Olympic Mountain front, pass south of the towns of Port Angeles and Sequim, around Sequim Bay, and leave the county on the Miller Peninsula (Fig. I-1).

The purposes of this study were to review TMPC's assessment of possible impacts on groundwater resources in Clallam County, and to provide the county government with an independent opinion on potential impacts, and what might be done to avoid or ameliorate them. The present work is intended to act as a review of the subject, and is not intended to serve as a complete groundwater study or environmental impact assessment (the former is being prepared by the U.S. Geological Survey; the latter is the responsibility of the applicant). Rather, this report contains a general description of the geologic and hydrogeologic conditions, and provides a summary of the kinds of information that will be necessary to assess impacts and make design decisions.

TMPC's certification application contains several references to geological conditions (Section 3), groundwater (Section 4), effects of oil spills (Section 4), and mitigation of adverse environmental impacts (Section 7). Most of this information is very general in nature, and inadequate for local design or decision-making.

The only published report dealing with groundwater in the county is Noble's (1960) report on the Sequim-Dungeness area. Unfortunately, the pipeline corridor passes south of his study area.

The major data source for our study was a print-out of water well data (depth, depth to static water level, location, etc.), compiled by the U.S. Geological Survey (Johnson & Rasmussen, 1980). These data were used to estimate depth to the water table at wells within about a mile of the pipeline corridor. The logs for most of these wells are on file with the Water Resources Division of the U.S.G.S. (Tacoma),

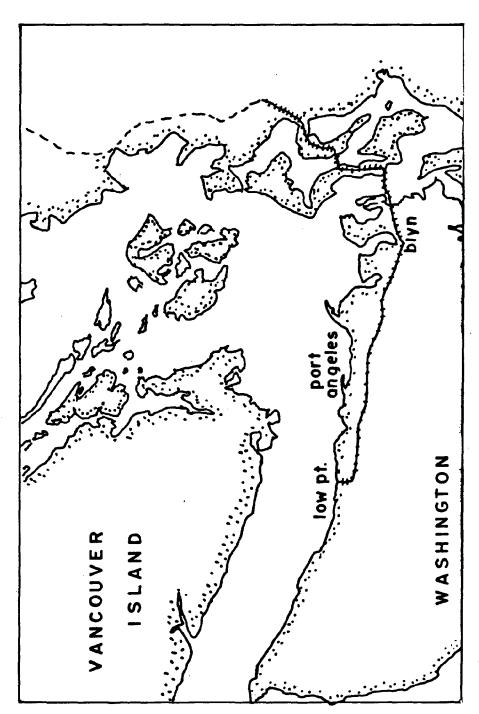


Figure I-1. Proposed pipeline route through Clallam County.

and with the Washington Department of Ecology (Redmond?). The logs were not consulted in this study.

Information on oil spills, oil migration, and groundwater contamination can be found in Freeze and Cherry's (1978) text, and in an article by Dietz (1971) on water pollution by oil. Other references are contained in these works.

Discussion

Hydrogeology of Eastern Clallam County

A. Geology

The area of interest extends from the front of the Olympic Mountains to the Strait of Juan de Fuca, and from Low Point to the Miller Peninsula (Fig. I-1). The northern edge of the Olympics are made of volcanic and sedimentary rocks (~25 to 65 million years old). Bedrock crops out in the hills, in river bottoms, and along parts of the coast. Most of the layers dip northward in the vicinity of the pipeline corridor [though a syncline (concave-upward fold) underlies unconsolidated sediments near the Strait; see the map of Tabor and Cady, 1978].

Overlying the bedrock in most of the study area is a layer of unconsolidated material, variable in thickness, lithology, and texture. These deposits include soils, glacial till, glacial outwash (loose sand and gravel deposited by meltwaters), lake sediments, bog deposits, and old weathered alluvium and slopewash materials. Young, loose alluvium is deposited on the bottoms of larger stream valleys. Because of the irregular nature of the bedrock surface and the processes that formed them, the unconsolidated deposits range in thickness from a thin mantle over bedrock hills to thick fills in the Sequim area.

B. Hydrogeology

Most of the groundwater tapped by wells in Clallam County is derived from the unconsolidated materials, especially gravel and sand units. These loose, permeable materials are irregular in shape and size (small lenses to larger layers). They are interbedded and interfingered with less permeable materials, which may partially perch or confine the aquifers. The hydraulic conductivities of these materials probably vary over ten orders of magnitude (i.e.,

 10^{-11} to 10^{-1} ft/sec), so flow rates are equally variable. Because of this irregularity, it is impossible to completely define the flow system, especially given the small number of wells in and near the pipeline corridor.

However, it is possible to draw some inferences about the flow patterns in larger scale. We may consider the groundwater system to be limited to the unconsolidated deposits, since groundwater in the bedrock is limited in volume, slow in movement, and (so far) insignificant in exploitation. Bedrock serves as the deepest surface over which most of the descending groundwater probably accumulates.

The flow system is also controlled by surface topography. Highlands, especially the Olympics, are recharge areas where water percolates predominantly downward. Lowlands, such as river valleys and the coast, are discharge areas, where water flows upward and out toward the surface. The water table is usually a subdued image of the ground surface: higher (but deeper) under hills, lower (but shallow) under the valleys (Fig. I-2). Depending on the details of local topography and stratigraphy, there may be small local flow systems, as well as a regional system (see Freeze and Cherry, 1978, Chap. 6).

In northern Clallam County, the dip of the bedrock and the general northward slope of the land combine to cause northward flow in the regional system, and a general northward decline in water table elevations (Noble, 1960). However, local flow may be in any direction if it is controlled by a stream valley or a hill. In general, shallow (saturated) flow follows the surface topography, and deeper flow is predominantly northward.

C. The Pipeline Corridor

The TMPC corridor is shown on Figure I-1, and on the topographic maps accompanying this report (see the Appendix). Along most of its length, the pipeline would be very near the contact between bedrock and unconsolidated deposits (as mapped by Tabor and Cady, 1978). The covering of soil, till, etc., is extremely variable in the hills, so the depth to groundwater also varies significantly.

Data from the U.S.G.S. print-out were used to plot depths to static water levels (SWL) for wells near the corridor. The SWL is not necessarily the water table--in wells that are closed except near the bottom, the SWL may only

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LOCAL FLOW SYSTEMS

Figure I-2

Schematic diagram of local and regional flow systems in northeastern Clallam County. The water table (dotted line) is a subdued image of the surface. Hypothetical flow lines are shown by arrows.

reflect the pressure in the aquifer being tapped (the piezometric head), and have nothing to do with the water levels nearer the surface. Therefore, water levels that are probably indicative of water table depth (in shallow or open wells) are plotted on the topographic maps in blue, others (which may or may not indicate water table depths) are plotted in red. There are not enough points to define a water table surface, but some general comments can be made based upon these data.

From Low Point, the pipeline corridor trends southeast to State Highway 112, and then parallels the highway on the south. For the first 4 miles it crosses flat to gently-rolling topography. There are no wells in this stretch. However, bedrock is exposed in the valleys of Field Creek and Whiskey Creek, suggesting that the deposits are 50 to 150 feet thick. Water levels are probably less than 50 feet deep.

In the 2 miles between Joyce and Ramapo, the corridor crosses slightly steeper terrain (about 10% slope gradient), underlain by sandstone and siltstone. Groundwater is probably present at shallow depths (10 to 20 feet ??), but there are no wells in this area.

From Ramapo to the Elwha River (5 miles), the pipeline would cross flat unconsolidated deposits between bedrock hills, then over a bedrock saddle and into the Elwha Valley. Again, groundwater depths should be shallow on and near bedrock. West of the Elwha, SWL's in wells near the corridor are mostly 25 to 45 feet, though two wells have SWL's of 4 and 5 feet.

From the Elwha, the corridor crosses U.S. 101, then hugs the mountain front for 2 miles, all on colluvium, alluvium, and glacial deposits. Slope gradients are commonly up to 20%. The water level is very shallow (1 to 5 feet) in some wells, deeper (10 to 60 feet) in others. The corridor then leaves the hills, and crosses rolling plains through the southern outskirts of Port Angeles. Here the deposits are up to about 300 feet thick, and SWL's are 30 to 120 feet deep. The pipeline would have to cross five perennial streams in the vicinity of Port Angeles, and would therefore be buried at or below the water table in those valleys.

From the east end of town, the corridor crosses a thick fill of unconsolidated material for about 8 miles. The terrain is gently rolling, and incised by several streams which have cut channels 100 to 120 feet deep, creating local flow

systems near themselves. There are some springs, shallow marshes, and ponds, suggesting that groundwater is reaching the surface on the plains as well as in the valleys. SWL's that represent the water table are 13 to 33 feet deep; piezometric levels are somewhat deeper, 28 to 70 feet.

On the north edge of Lost Mountain, the corridor nears the mountain front, then crosses alluvial fill of the Dungeness valley. Water levels are fairly shallow in the low hills (mostly 6 to 30 feet), but deeper on the edge of the Dungeness Valley (60 to 80 feet), presumably because the water table is depressed along the incised valley.

From the Dungeness River, the pipeline corridor stays in the foothills around Burnt Hill (made of basalt, as are Lost Mountain and Bell Hill), south of Sequim Bay and eastward to the Jefferson County line. The mountain front deposits (colluvium and till?) are moderately steep (5 to 50% slopes) and crossed by many forest streams. The stratigraphy of these materials is extremely variable, so the water levels are as well (2 to 400 feet deep).

D. Information Needed

The generality of the above discussion suggests that further study and information are necessary. Specifically, examination of the logs of wells along the corridor would provide a more detailed picture of the stratigraphy of deposits in the area. Soils data (Clallam County soil survey?) might provide the infiltration capacities of surface materials. With these data and with reasonable estimates of the hydraulic conductivities of subsurface materials, it should be possible to do a better analysis of flow paths and water table configuration, at least in the areas with wells.

At greater expense, it might be necessary to dig new wells in some areas to get the required information. Tracer studies, computer models, or other techniques might also be employed. A hydrogeologic consulting firm with experience in these areas would be required in order to plan these studies.

Hydrogeologic Effects of an Oil Spill

If a pipeline leak should occur, the amount of oil that would get into the ground would depend upon the rate of leakage, the viscosity of the oil, and the permeability of the trench fill and the ground around the trench. If it entered the soil, the oil would percolate vertically through the unsaturated zone, leaving a cylinder of residual oil held in the soil pores by capillary tension, and adsorbed onto the grains (Dietz, 1971). The oil would spread out laterally on reaching layers of lower permeability. This downward percolation process may totally exhaust the oil before it reaches the water table, depending on the depth of the latter.

If the oil reaches the water table, it will spread out in the capillary zone, since the oil is lighter than and immiscible with water (Fig. I-3). This oil will "pancake" until enough of the oil is adsorbed or held by tension to make the oil immobile.

Van Dam (1967) provided a simple method of calculating the depth of oil penetration, and the size of the pancake that will form if it reaches the water table. The method is based upon the volume of oil spilled, the area over which it spreads, and the physical characteristics of the oil and the soil. In order to apply this model, specific data for different sites and estimates of leakage rates are necessary. A general conclusion that can be drawn from this model is that a given volume of oil will spread through 10 to 20 times that volume of soil.

The environmental impact assessment should include reasonable estimates of depth of penetration for conditions along the corridor, based upon Van Dam's method or some other model. In addition, rates of percolation should be estimated for local conditions. These kinds of information will be necessary to determine whether a given water table depth is really safe from quick contamination.

Even though a slug of leaked oil becomes immobile, the effects of the spill may extend much further. Rainwater percolating through the residual oil will take into solution the lighter fractions of the oil, and carry them into the groundwater system (Fig. I-4). Eventually the lighter components will all be washed out, leaving an immobile glob of inoffensive paraffinic oil. However, because the solubility of the lighter components (20 to 80 mg/l) is so much greater

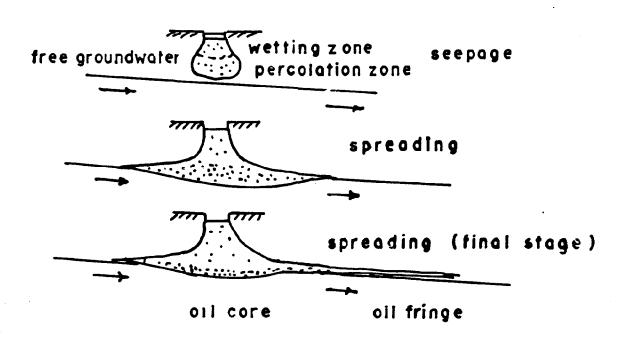


Figure I-3 Stages of migration of oil seeping from a surface source (Freeze and Cherry, 1978).

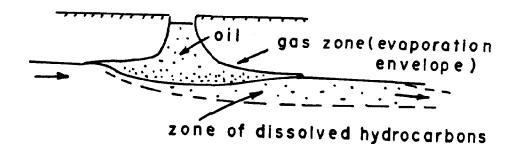


Figure I-4 Migration of dissolved and gaseous hydrocarbons from a zone of oil above the water table (Freeze and Cherry, 1978).

than the amount necessary to affect the quality of water (0.005 mg/l gasoline in water can be tasted), the contamination can spread over a large volume of groundwater, and last for a long time. Processes of dilution, dispersion, adsorption, oxidation, and anaerobic reactions will lower the concentration of contaminants in time, but the "time" may be measured in years (Dietz, 1971).

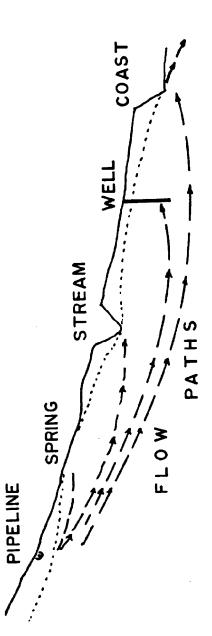
Once in the groundwater flow system, the oil components will move in the same direction as the water (Fig. I-5). If it is in a local flow system, it may move at shallow depth toward a small stream or spring, and emerge at the surface. If it gets into the regional flow system, it may move through deep aquifers to a large river or emerge at the coast. Like the groundwater movement itself, the migration of oil-contaminated water depends on the details of local stratigraphy, permeability, and flow. Impermeable layers may protect underlying aquifers, or cause concentration of the polluted water near the surface, or both.

According to the TMPC application, a rupture of the pipeline could cause up to 11,320 bbl (1800 m^3) of oil to leak, at an initial rate of 56 ft^3 /sec (Section 4.3.3). This amount could not all percolate into the ground if the rupture is in a buried section, but could spill onto the ground in an above-ground section, and then percolate through the surface.

Surface oil could be contained and salvaged. If preservation of the soil is important, the containment area should be small, however, this will cause deeper penetration of the oil. If the water table is shallow and protection of the groundwater is important, the containment area should be large, so the depth of penetration can be minimized (Dietz, 1971).

Contaminated soil, especially in the pipeline trench, can be dug out and cleaned or replaced. Shallow wells might be used to pump out part of the oil that is still mobile, but this is not a realistic solution.

It may be better, especially in sensitive areas, to prevent any infiltration by lining the trench with impermeable materials (applicant has acknowledged this idea in Section 7.1.1.1). The trench might be provided with some kind of drainage system to collect and remove oil leaked from the pipe.



Possible flow paths of soluble oil components. Could reach springs, streams, wells, or the coast, depending upon the local flow system. Figure I-5

If spilled oil does cause contamination of groundwater, it may be necessary to install wells downstream of the ruptured section. One set of wells could capture contaminated water for treatment, and another set could recharge the aquifer with clean water (if economically or environmentally necessary) (Fig. I-6).

Conclusions

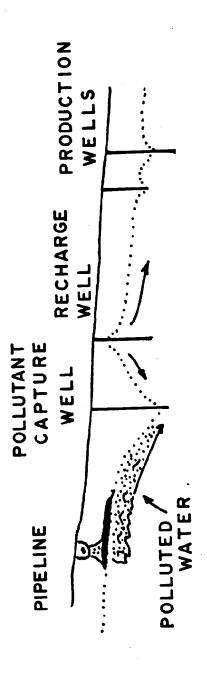
If the pipeline is built so that leakage of oil into the ground is possible, it may be necessary to designate areas in which more stringent (and expensive) design measures should be taken to protect the groundwater resource. Such "sensitive areas" might include:

- Areas in which a spill would reach the water table in a short time (i.e. too short to be dug out);
- 2. Areas in which the oil or its soluble components could return to the surface to pollute surface waters important to man, agriculture, or fisheries;
- 3. Areas in which the light components of spilled oil are likely to pollute aquifers important to water wells.

Obviously the decisions on sensitive areas and design criteria in them must be based on information on the conditions of the flow system, the likelihood of pollution of aquifers, and the costs of prevention and/or amelioration of spill effects. TMPC's application gives no usable information on these factors.

Recommendations

The groundwater geology and hydrology of the TMPC corridor in Clallam County is very poorly known despite the importance of the resource in the eastern part of the county. Much more information is needed to adequately evaluate the potential impacts of pipeline construction or an oil leak on the groundwater system. If TMPC reactivates its application, it should be required to provide specific data on flow paths, water table depths, and flow rates of leaked oil.



Possible arrangement of wells to capture oil-polluted groundwater, and recharge the system with clean water. Figure I-6

SECTION II: SEISMICITY

Introduction

This section provides epicentral maps for Western Washington and for the Puget-Vancouver tectonic province. Two sets of maps are provided, the first of which displays all known historic events through 1980 for these two regions (Figures II-1 and II-2), and the second of which shows all events recorded during 1980 (Figures II-3 and II-4). The data plotted include events from magnitude O to 7, and represent the most complete available compilation of earthquake data for Western Washington. A detail map showing the epicenters of the February 14, 1981 Elk Lake swarm is also provided. The applicant's seismic design analysis is discussed in light of the seismicity maps.

Discussion

A. Seismicity

Figure II-1, the historic epicentral map for Western Washington, shows a high degree of seismicity from throughout the Puget Sound Trough and north through Vancouver Island. The distribution of these earthquakes, in a spatial and temporal sense, appears to be random in the Puget-Vancouver Island region. The spatial density of earthquakes does decrease in a westward direction along the Washington-Juan de Fuca coastline from just west of Port Angeles to Pillar Point, where seismicity again increases. The significance of this "quiet zone" is only marginal, since there is moderate seismicity to the north and south of the coastline just 20 km away on either side. The proposed TransMountain Pipeline route passes along this zone and then into a zone of high seismicity east of Port Angeles.

Figure II-2 provides an expanded view of the seismicity in the Puget-Vancouver region, using the same data base (included as Appendix II-1) as is mapped in Figure II-1. This figure demonstrates that the seismicity in the region around Clallam County cannot be unequivocally segmented into zones of high and low seismic risk, since earthquakes have clearly occurred over the entire area. Arbitrary demarcation of two zones within Clallam County, which the applicant

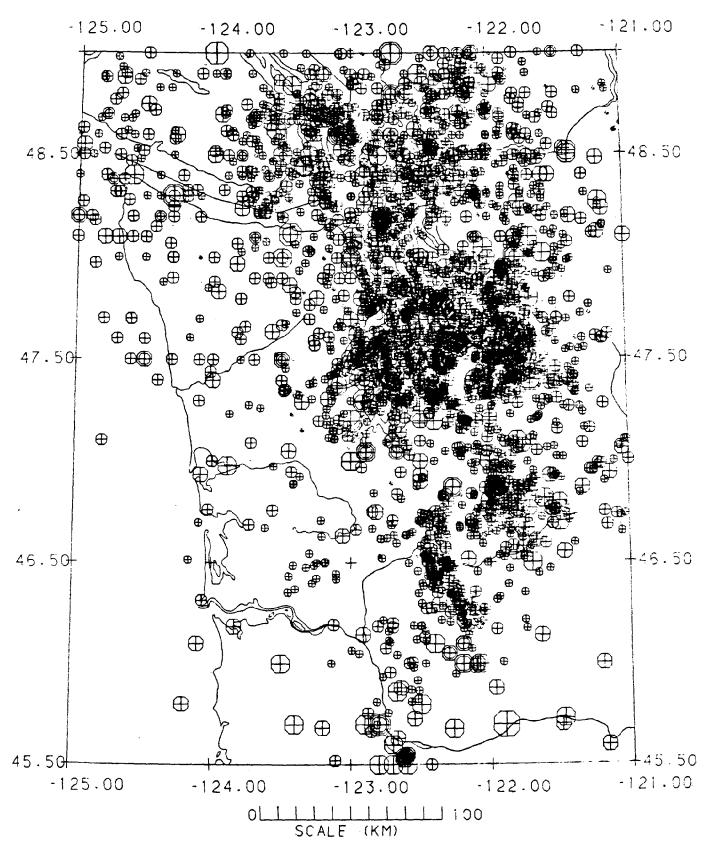


Figure II-1 Historical seismicity in Western Washington. Increasing symbol size indicates greater magnitude. Largest event 7.2 M, smallest 1.5 M.

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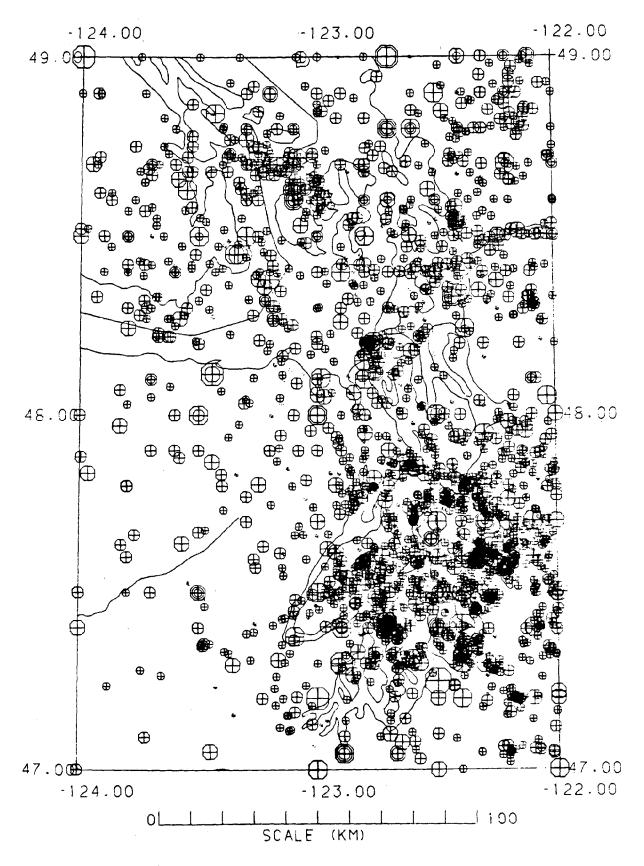


Figure II-2 Historical seismicity of the Puget Sound-Vancouver Island region.

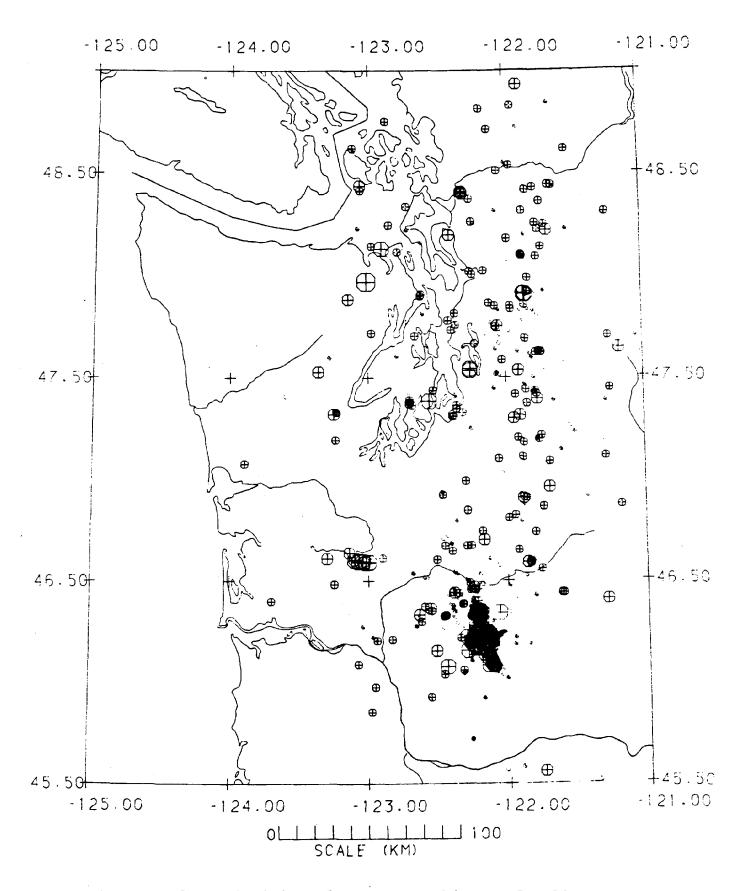


Figure II-3 Seismicity of Western Washington for 1980.

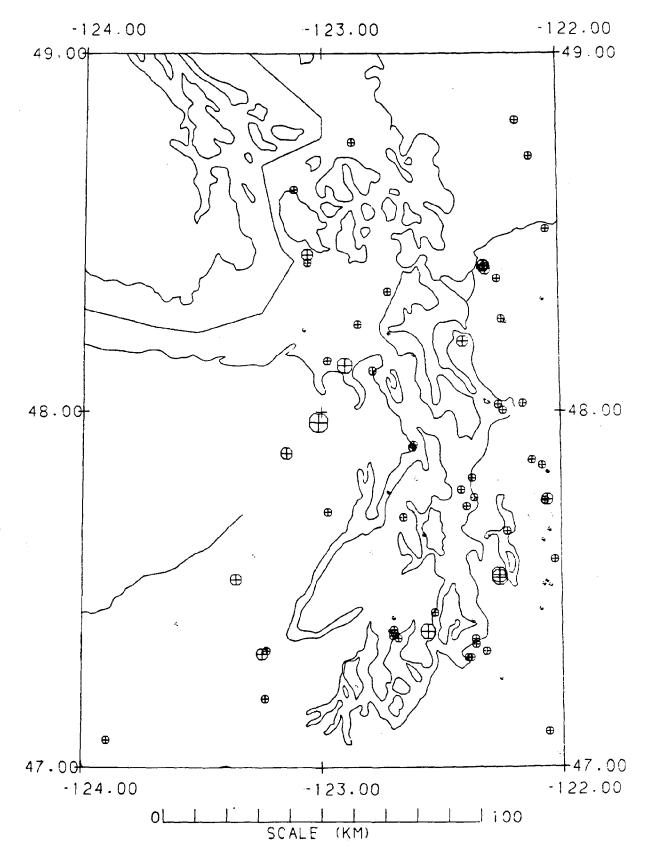


Figure II-4 Seismicity of Puget Sound-Vancouver Island Region, 1980.

has done in its seismic risk analysis (Fugro, 1980; and reproduced as Figure II-5 in this text), is not satisfactory. Comparison of the lower risk Zone A in Figure II-5 with either Figure II-1 or Figure II-2 illustrates this point.

B. Shallow Earthquakes

Depth-magnitude relationships (the correlation of hypocentral depth to the earthquake magnitude) have until recently indicated that large Puget Sound earthquakes (5.0 M and greater) occur only at depths exceeding 40 kilometers. On February 14, 1981 a shallow (4 km) earthquake occurred at Elk Lake. The main shock had a magnitude of 5.5, and was followed by over 300 smaller aftershocks.

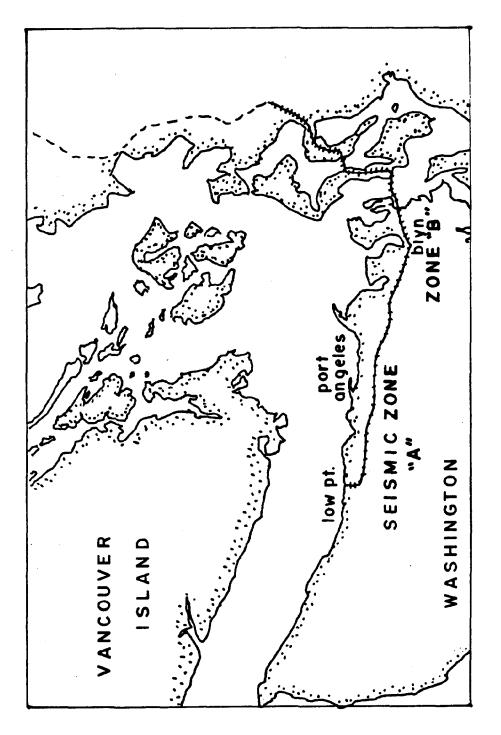
A plot of all events greater than Richter magnitude 1.0 is presented in Figure II-6. The data used in this plot were obtained from digital recordings obtained at the Geophysics Program of the University of Washington. The University of Washington state network includes more than 100 active stations, all of which telemeter their seismic information in real time to the central recording computer located at the Geophysics Program laboratory.

The Elk Lake swarm was recorded by a majority of the network stations, providing excellent azimuthal control and good depth control. Although the depth of the initial event and the following swarm was shallow (between 4 and 10 km), the accuracy of the depth estimates is still reasonably good, and is comparable in quality to an intermediate or deep earthquake. The reason for increased potential error is that the Puget Sound Seismic Model has only a single layer for the top 5 km of the crust, and that the close-in seismic stations, which would provide the best control for depth in a shallow event, were overloaded by the large events. Estimated average depth accuracy is ± 0.5 km.

The importance of the Elk Lake event is that it demonstrates that large, shallow earthquakes can occur in Western Washington. This, coupled with the random spatial distribution of earthquakes in the region, indicates that critical facility design in Clallam County should consider the potential for large shallow events.

C. Attenuation Relationships

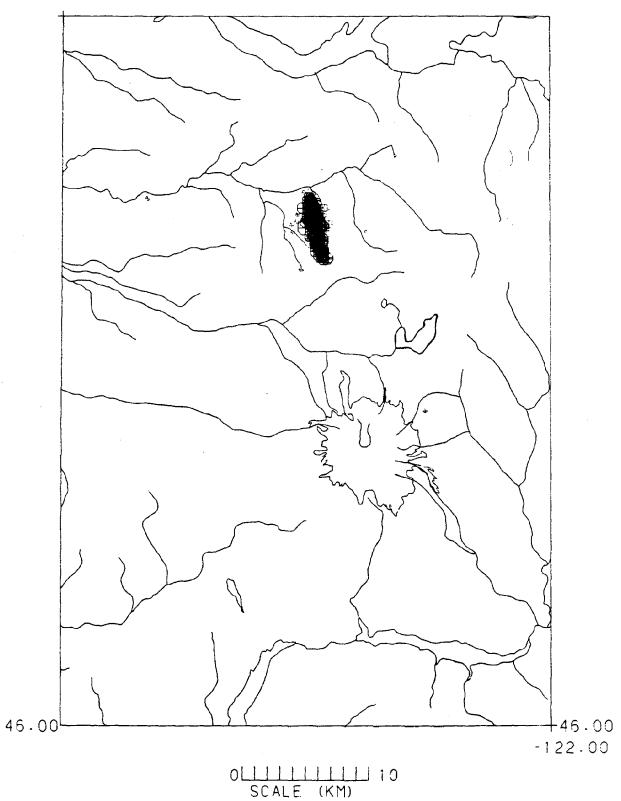
The evaluation of the seismic quality factor Q is an important part of an earthquake risk evaluation for a



proposed seismic applicant's Map indicating zones A and B. Figure II-5

Figure II-6 Elk Lake Earthquake Swarm (Feb. 14, 1981). Elk Lake is 18 km North by NW of Mt. St. Helens.





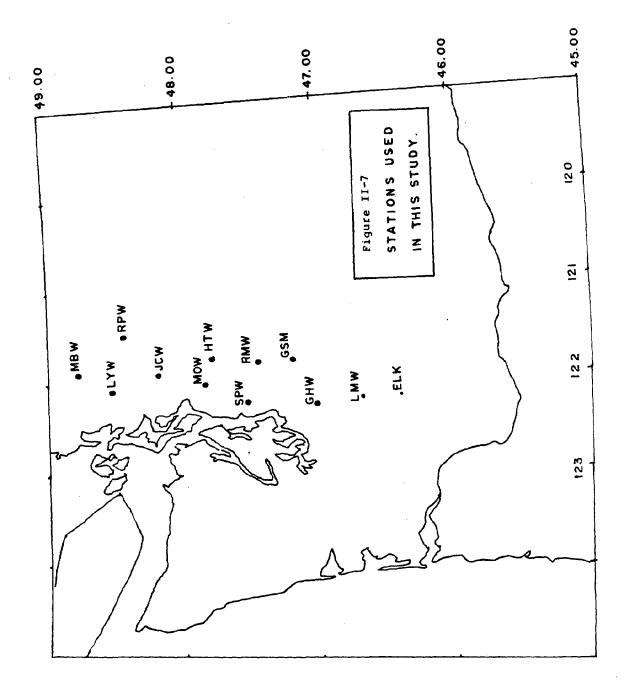
particular region. An understanding of the degree of seismic dissipation can allow more accurate modeling of potential ground motion and thus provides greater insight into the problem of damage prediction and design criteria for critical structures.

The estimation of Q for compressional waves traveling through the upper crust in the Puget Sound region represents the first concerted effort at measuring the attenuation characteristics in this area. Previous work has been only on a regional or continental scale. Solomon and Toksoz (1970) found that the northwest U.S. was located in a band characterized by higher P wave attenuation, relative to the California coast and the Great Plains region. It should be pointed out that the resolution of their measurement is a dimension equivalent to the combined widths of Washington and Oregon and thus is somewhat irrelevant. Langston and Blum (1977), however, also found that the region was characterized by high attenuation (Qp = 65) using teleseismic Pp data. Langston (1981), in a short review of Langston and Blum (1977), indicated that the low Q may be due in part to scattering effects, implying that the Q value should be higher than reported.

Data

The data used in the calculations were from selected events of the Elk Lake swarm. Several hundred events occurred within a few days after the main shock of February 14, and out of these events were picked for quality of location, impulsive signature, high signal-to-noise ratio, and maximum range of receiving stations.

A map of the stations used in the study is shown in Figure II-7. Also included in the figure is the location of the Elk Lake swarm. The geometry of the station array is that of a line bearing north up the axis of the Puget Trough. Elk Lake is at the southern-most end of the line. This geometry also allowed for a velocity model check since it represents a single-ended refraction line.



Method of Analysis

Three methods were considered for this study: the maximum sustained peak method, the Aki-Chouet scatter-coda method, and the spectral ratio technique.

Initial calculations using the sustained peak method showed significant scatter and proved to be less consistent than required. The reason for this is not immediately at hand; however, the problem of multipathing, scattering, and topography may combine in the Puget Trough to cause significant amplitude fluctuations within the area.

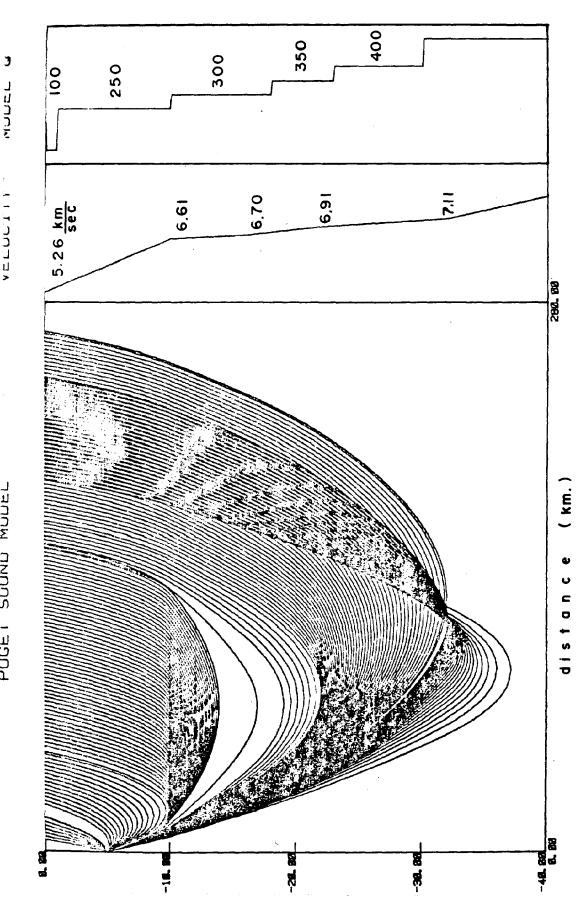
The Aki-Chouet scatter method was not attempted due to certain technical requirements of the technique which were not met by the network. Specific conditions must be met between seismometer bandwidth and source frequency. The method is important, however, since it is least sensitive to the problems of scattering, topography, geometric spreading, and the like.

The spectral ratio technique, as used by Teng (1968), Solomon and Toksoz (1970), Solomon (1972), and others, has become as widely accepted a technique as the peak-sustained method. It has as an advantage that it can be used with seismic records that even have minor clipping.

Revised Attenuation Parameter

The spectral ratio technique was used in this study in association with a forward-modeling program using seismic ray tracing. Although the structure of the Puget Trough is acknowledged to be heterogeneous, the velocity models available for use are all flat-layered ones. This is because only limited refraction data are available in the region and an adequate two-dimensional model does not exist at this time. Ray tracing was conducted using a velocity model refined from the standard Puget Sound seismic model.

The results of this study are summed up in Figure II-8. This figure provides the actual ray trace sequence for the refined model, the velocity model for the Puget Sound region, and final computed multilayer Q model for the region.



Combined ray trace, P-wave velocity model, and Q model for the Puget Sound lowlands, using the Elk Lake events as the seismic source. Figure II-8

The Q model reveals a Q of 100 for the first two kilometers of the crust, followed by a continually increasing Q with increased depth. The average Q value for mid-crust (~15 km) is 300 ± 50 ; this corresponds to an attenuation coefficient of 7.87 x 10^{-3} for this depth.

One conclusion of this study is that the Puget Sound region is not an area of high seismic attenuation. Q values of 65, as reported by Langston (1981), may be appropriate for near the surface, but do not correspond to the transmission characteristics at depth.

This Q model and its corresponding attenuation coefficient has been measured in the Puget-Vancouver region, and as such represents an appropriate attenuation relationship for use in a seismic design analysis. The applicant (Fugro, 1980) does not use in its design analysis a local or regional attenuation relationship, but rather it uses one developed for Southern California earthquakes. The applicant does not demonstrate that the earthquakes and structure of the Puget-Vancouver region are similar to the earthquakes and structure of Southern California, a discussion which might have laid an argumentative foundation for using the California attenuation relationship.

Conclusions/Recommendations

Seismicity in the Puget Sound-Vancouver Island region is high. The northern portion of the Olympic Peninsula has a high-to-moderate seismicity with no obvious changes in earthquake distribution which might be used to identify zones of lower seismic risk. Although the large historic earthquakes have been deep-focus events, the Elk Lake event demonstrates that large shallow earthquakes will occur in the region. Seismic risk calculations must therefore consider the Clallam County area as one homogeneous seismic zone, and must take into account the potential for shallow events. The applicant uses a two-zone calculation, which is not acceptable. The basis for this rejection is not one of competence, but is primarily due to the use by the applicant an incomplete seismic data base. The applicant's analysis may be acceptable if Zone B, the higher risk area, is extended westward beyond Low Point, and an improved attenuation relationship is used. Utilization of attenuation formula appropriate for the Puget Sound region may not alter the results of the analysis, but usage of relationships developed for Southern California is not appropriate unless substantiated.

SECTION III: LOW POINT FACILITIES

Introduction

This section reviews the applicant's submitted materials which consider the geotechnical details of the proposed Low Point moorage and tank farm facility. The reports used in this section include Harding-Lawson, 1980 (#HLA 9053,012.01), Harding-Lawson, 1980 (#HLA 9053,013.04), and Fugro, 1980 (Appendix I). The Harding-Lawson reports emphasize the preliminary nature of their studies, and the present analysis takes this into account.

Discussion

The Low Point area is proposed for use as the location for tanker off-loading and for an oil tank farm facility. Required geologic/geotechnical information includes depth to bedrock, type of sediment cover, liquefaction potential, near-surface structure, and potential for mass-wasting (slumping, sliding, etc.). Engineering properties of soil and bedrock will not be discussed.

High frequency seismic profiles were made by the applicant just offshore from Low Point (#HLA 9053,012.01). The applicant has submitted only interpreted line drawings of these data. A general explanation of the high frequency reflection technique and the types of equipment used, and a discussion of measurement error is provided in Appendix III-1 of this report.

For the most part, the area is characterized by mild topography and long slopes. Minor structural outcrops occur. Sea bottom consists of consolidated sedimentary rock at the surface, or is overlain by 1 to 3 feet of sediments. No deeper structure (except at Whiskey Creek) was interpreted from the data. The Whiskey Creek structure is a small sediment-filled basin.

Liquefaction damage in the offshore area will be minimal because of the very shallow depths of sediment. Structures located offshore would have a foundation in consolidated rock, and if liquefaction did occur only the first

few feet of material could slough away. Assuming that this potential is considered in the foundation design, liquefaction will not pose a significant problem.

Offshore slumping in this area also plays a minimal role. Although slopes in local areas exceed 5°, generally the topography is flat with a thin sandy cover. Slumping will be minimal in this area.

Exploration efforts at the onshore portion of the Low Point facility was restricted to a reversed seismic refraction line and two soil borings. The borings reveal silty clay soils overlaying siltstone bedrock. Excavation to consolidated bedrock is planned in the tank farm design, hence liquefaction will not be a problem at the tank facility. The refraction lines show 10 to 20 feet of low velocity material (the silty clay) over bedrock, dipping north down to the beach area. During periods of high rainfall this top layer will become partially saturated and groundwater will follow along the bedrock contact. Slumping of the bluffs along the beach at Low Point may be enhanced by this flowage.

Recommendations

Liquefaction damage potential for the Low Point area is small if structure foundations are located on bedrock. Two soil borings represent reconnaissance and not exploration, and several borings must be made to adequately characterize the tank farm facility.

Submarine slumping does not pose a great problem at the site. Slumping rates of the bluffs onshore may be changed with major construction at the tank farm site, since moisture control will be required for engineering purposes (#HLA 9053,013.04). Flow of petroleum products or contaminated water, if injected into the soil layer, will flow along the bedrock contact and drain down the bluffs. Although this represents significant contamination to the local groundwater and bluffs, it is fortuitous that the runoff will not flow into a major aquifer.

Structurally, the Low Point site does not have any significant problems. More detailed work is needed, however, in the form of soil borings as mentioned above.

SECTION IV: ANCHOR PENETRATION

Introduction

A significant problem in the deployment scheme of a pipeline is to provide adequate protection from anchor penetration. Penetration may occur from a direct vertical drop of the anchor from a ship, or from the intersection of the pipeline with an anchor which is being dragged along by a ship. Anchor drag distance may extend several thousands of feet, depending upon anchor style and weight, ship size and inertia, and sea conditions. Penetration depth into the sediments along the submarine crossings also will vary considerably as a result of the above conditions, as well as with sediment type, sediment strength, seabottom topography, and shallow structure below the mudline.

The anchor penetration problem has been discussed by the applicant in Section 3.3 of Harding-Lawson report #HLA 9053,017.04.

Discussion

The evaluation of anchor penetration depth can be perseveral ways. Calculations using measured in strength data from the soil types under question should be made, regardless of the final method used for depth and local estimation, since local sediment type and depositional characteristics will have an effect on bulk strength. applicant's discussion of anchor penetration consists of a presentation of a table of anchor penetration depths for several anchor sizes, for two anchor styles, and two sediment types. The table is reproduced here as Table IV-1. The applicant does not discuss local sediment variations nor the possibility of strength variations within a single sedi-The applicant also does not provide laboratory ment type. strength data to compare with the applicant's standard table. Their table provides penetration depths for two sediment types, using the terms "mud" and "sand". Mud may be defined as a mixture of water with clay and/or silt, plus minor miscellaneous materials such as organic debris, erratic material, etc. Sand may be defined as detrital material of size range 2 to 0.06 mm in diameter.

TABLE IV-1

ANCHOR BURIAL DEPTHS
(FROM VALENT AND BRACKETT, 1976)

| · | Fluke Tip Burial Below Bottom | | | | |
|---------------|-------------------------------|--|-----|-----------------|-----|
| Anchor Weight | Standard Stockless | | | Danforth or LWT | |
| | Sand | | Mud | Sand | Mud |
| 1b | ft | | ft | ft | ft |
| 3,000 | 3 | | 7 | 8 | 23 |
| 10,000 | 5 | | 11 | 9 | 28 |
| 20,000 | 6 | | 12 | 10 | a |
| 30,000 | 7 | | 17 | a | a |

a_{No data}

applicant suggests mapping the submarine crossing into areas of mud and sand, and using the table to estimate depths. However, the materials typical of the crossings will always have some percentage of both mud and sand. The applicant does not suggest a provision in its procedure for this unavoidable condition. It is conceivable that significant underestimation of penetration depths could be made if a sandy mud or a silty mud is classified as "sand". The table also does not list penetration depths for 30 ton anchors, which would be carried by 300,000 dwt tankers. The applicant does not discuss the problem of anchor drag and subsequent variations in drag depth.

Recommendations

The concept of using a standard table for determining anchor penetration depths is satisfactory if provisions are made for the variability in sediment composition which exists along the submarine crossings. Acceptance of design penetration depths should be made only after adequate mapping and strength tests have been performed. Penetration depths for 30 ton anchors should be included in the analysis. Maximum depth of penetration for anchors dragging through the sediments must also be submitted. A reference depth should be used in the design; for example, set the pipeline burial depth to be four feet below the computed penetration depth for the largest anchor to be frequently used by ships crossing the route. This provides a maximum continuous protection for the pipeline and provides a built-in safety margin. The applicant's discussion of anchor penetration is minimal and does not set forth the true design depths, which could approach depths of 20 feet or greater. The applicant's table of depths must be accompanied by a map detailing the location of regions which are made up of "mud" and "sand" and of mud-sand mix.

SECTION V: SUBMARINE CROSSINGS

Introduction

This section reviews and critiques the materials submitted by the applicant which consider the submarine portion of the proposed pipeline. The primary documents reviewed are HLA 9053,012.01, HLA 9053,002.01, and Fugro, 1980. Submarine slumping, structure and sediment liquefaction potential is evaluated. Anchor penetration has been discussed in Section IV.

Discussion

The purpose of the reports referenced above was to obtain geologic, geophysical, and geotechnical information about the bottom and sub-bottom sea floor along the Oak Bay, Admiralty Inlet, and Saratoga Passage pipeline route corridor. The data gathered consists of a small sequence of Vibracore samples and Vibracore jet tests, elementary physical property tests on the Vibracore samples, and continuous bathymetric, high frequency seismic and side-scan sonar profiles.

The seismic source used was an electro-mechanical type transducer which generated frequencies in the 0.5 to 2.0 kHz band. This frequency range is adequate for shallow penetration of the bottom sediments and has the potential for good resolution (see Appendix III-1 for an explanation of different seismic sources, their typical depth of seismic penetration, and typical resolution). The high resolution of the seismic source, coupled with the high resolving properties of the side-scan sonar, should have provided high quality profiles of the sub-bottom; however, the interpreted line drawings submitted by the applicant show a fairly The form of data presentation (short segments resolution. of profiles on single pages) is also poor, and is not conducive to integrated interpretation. The profile interpretations do not discuss faulting or possible faulting, nor is there a discussion of why some of the seismic reflectors mapped in the interpreted profiles suddenly are truncated (Plate 13B, reference HLA 9053,012.01). Abrupt truncation of a series of reflectors is often indicative of faulting or

mass wasting. No references to regional faults and the possibility of active sub-bottom faulting are made in the applicant's discussion of the submarine crossings.

Slope stability along the submarine crossings has not been adequately addressed by the applicant. Slumping along slopes of less than 10 degrees have been caused by earthquakes equal to or lesser than the design earthquake of 7.5 (see Table 1 in Appendix V-1). The potential sediment volume in large slumps can be great (Table 2, Appendix V-1), and potential damage is greater if slump-initiated turbidity currents are generated. The geophysical profiles submitted by the applicant show that along the Oak Bay crossing slopes exceed 30% on the west side and 20% on the east side. Samples from the Vibracore station in Oak Bay indicate silty mud of low strength. Slumping potential is very high, as is sediment liquefaction (discussed below). Slumping along the Admiralty Inlet crossing may occur also, with western slopes of 8-10% and of greater than 10% on the east side. loose sediments are perched on these flanks of the inlet and could very well prove to be unstable, given the design The slumping potential along the Saratoga Pasearthquake. sage Corridor is greater than in Admiralty Inlet. Slopes to the west exceed 35 to 40%, and eastern slopes exceed 30%. Thin sediment layers are perched on these slopes, and will not be stable under the design earthquake conditions.

Liquefaction potential for all crossings has analyzed. The data used in this analysis include the Vibracore sample analyses and sample tests. The ground motion accelerations were that of Johnson and Rasmussen Relative densities of 60% were (1980).assumed Vibracore T values of less than 10 sec/ft. Liquefaction is the temporary loss of cohesion of a soil, caused by oscilla-This phenomenon occurs in the following tory motion. manner. When a saturated, low-to-medium dense sand is subjected to ground shaking, the material tends to compact and decrease in volume. This change in volume will in turn cause an increase in pore pressure, since fluid drainage is slow relative to the rapid loading of the volume. volume decrease causes a pore pressure that is equal to or greater than the overburden pressure (i.e., the intergranular stress becomes zero), then the soil has no strength and will physically become a flowing mud. The potential liquefaction is a function of the initial relative density of the soil, the degree of severity of shaking, and duration. In general, the probability of liquefaction increases as the relative density decreases, the shaking increases in severity, and the number of cycles (duration)

increases. Grain size distribution also plays an important role, with soils having a mean grain-size diameter of O.1 mm (very fine sand) considered most susceptible to liquefaction.

The procedure used to evaluate liquefaction potential was that of Seed and Idriss (1970), which is generally accepted as the most reliable of liquefaction computations.

The potential for liquefaction for a given soil type can be defined as the ratio of the earthquake-induced stress in the soil, $\tau_{\rm e}$, to the stress c required to initiate liquefaction. A $\tau_{\rm e}/\tau_{\rm c}$ ratio greater than one indicates

potential liquefaction of the soil.

Calculation of the earthquake-induced stress can be made by the following relationship:

$$\varepsilon_{e} = 0.65 r_{o} \frac{a_{max}}{g} r_{d}$$

where r_0 is the overburden pressure at the specified depth, a_{max} is the maximum ground surface acceleration, g is the acceleration due to gravity, and r_d is the soil deformation coefficient, determined experimentally.

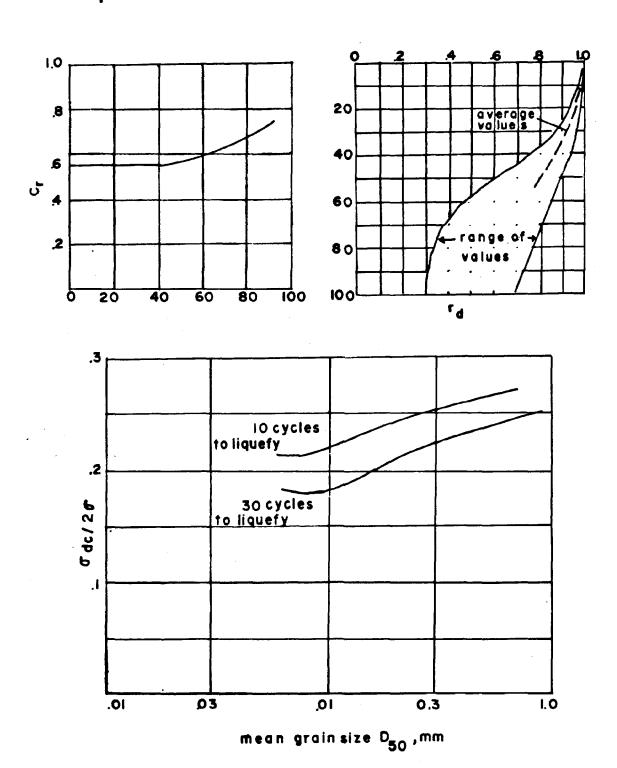
Calculation of the earthquake-induced stress can be made by the following relationship:

$$\varepsilon_{c} = \sigma_{eo} c_{r} (\frac{\sigma dc}{2\sigma a}) \frac{D_{r}}{50}$$

where $\sigma_{\rm eo}$ is the effective overburden pressure at the specified depth, $C_{\rm r}$ is a correction factor for laboratory data, $D_{\rm r}$ is the relative density, and $(\frac{\rm odc}{2\sigma a})$ is a stress ratio determined from dynamic triaxial soil tests.

The relationship defining the variables in these two equations are evaluated by Seed and Idriss (1970) from numerous previous studies, and are presented in Figures V-la, b, and c.

figure ∇ -la,b,c. liquefaction curves of Seed-Idress (1970)



The results of the analysis show that the potential for sediment liquefaction for Oak Bay and for portions of Saratoga Passage and Admiralty Inlet are very high when considering the maximum 7.5 M earthquake. Liquefaction potential under the 6.5 M probable event is significant, although the stiffer sandy silts will most likely not liquefy. The greatest potential for mass wasting is along the steep flanks of each of the crossings.

Conclusions

Mass wasting along the submarine corridor is a distinct probability, given the design 7.5 and 6.5 earthquakes. Slumping and sediment liquefaction will most likely occur along the steep slopes on the east and west sides of each of the crossings. Details of sub-bottom structure are not available.

Recommendations

More detailed seismic profiles are needed to adequately evaluate the sub-basement structure. Additional sediment sampling and sediment tests are needed to detail liquefaction problem. Contingency plans or alternate routes should be submitted in order to mitigate the slope slumping problem.

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APPENDIX II-1

EARTHQUAKE DATA USED IN SEISMIC STUDY

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APPENDIX III-1

CHARACTERISTICS OF MARINE SEISMIC SOURCES

by

D. M. Johnson

Characteristics of Marine Seismic Sources

Introduction

"High resolution continuous seismic reflection" (or continuous seismic sounding) is the widest-used and most economical method for studying the first hundred metres of soil beneath the sea floor.

The method enables the geometry, structure and configuration of the geologial strata to be determined. However, in the prevailing state of techniques, seismics alone does not make it possible to make any affirmation:

- as to the nature of the soils,
- and yet less, as to their physical and mechanical properties.

While certain interpretations sometimes justify a presumption as to the state of consolidation of the soils (owing to the degree of penetration, for instance of signals with a given frequency and energy), these assumptions must necessarily be verified by core samples or in situ geotechnical measurements.

Preliminary recording of seismic profiles on a marine site makes it possible:

- to fix the locations of the geological and geotechnical soundings (drilling/core drillings and in situ measurements) as a function of the variations in the configuration of the subsoil,
- to reduce the number of these soundings,
- to extrapolate where necessary the results of core drillings and in situ measurements.

All seismic techniques currently applied for the reconnaissance of marine soils use the continuous reflection method. The refraction method is applied only when seismic reflection proves to be inoperative or the results obtained do not yield the expected accuracy.

Several types of devices are used in "high resolution seismics." The main of them are:

- sediment sounders (or echo sounders)
- boomers

- sparkers
- side scan somar

These devices are characterized by their transmission frequency and consequently the penetration of the signal and its resolving power (or definition):

- the penetration is inversely proportional to the transmission frequency,
- the resolving power (and relective quality) decreases with the penetration and increases with frequency.

Since "Boomer", Echo Sounders, and Side Scan Sonar was used in the Shannon-Wilson reports, a discussion of their characteristics has been included in this Appendix.

BOOMERS (AND THE UNIBOOM)

The boomer or thumper is an electromechanical source invented by EEG.

Principle and characteristics of the boomer

Principle of the boomer

The boomer consists of:

- an induction coil against which an aluminium plate is applied by a system of springs,
- a bank of capacitors (connected to a sparking circuit) producing electrical discharges through the coil at regular intervals.

With each discharge, the eddy currents induced in the conductive plate cause it to move violently away from the coil. The initial movement of the plate triggers the acoustic pulse.

Characteristics of the boomer and Uniboom

The acoustic signature of a 1,000 J boomer has a signal duration of about 5 ms.

The spectrum for this boomer ranges from 200 to 2,000 Hz.

From the standpoint of enery distribution, the figure reveals:

- a very high amplitude of the initial pulse peak (a),
- a peak of negative amplitude (b) extending the signal.

This secondary peak is caused by the cavitation which arises behind the plate in the depressurized zone.

In the Uniboom system, the secondary pulse is eliminated by providing an elastic diaphragm on the inner face of the plate from the depressurized side. This diaphragm then absorbs part of the enrgy and thus limits the cavitation.

The duration of the Uniboom signal is limited to about 0.2 ms.

The frequency spectrum ranges from 500 to 10,000 Hz on the average (the frequency decreases slightly as the energy output increases).

The resolving power:

- of the boomer proper is not less than 2 m, owing to the considerable length of the signal,
 with the Uniboom, it can theoretically get down to 30-40 cm (comparable to the best sediment sounders).

Principle and equipment of the echo sounder

Principle of the echo sounder

The echo sounder puts out a brief ultrasonic pulse which is reflected from the sea bottom. The return echo is amplified and then continuously recorded.

Let V be the speed of sound in water and t the time interval between the emitted and return echo, the depth H is given by:

$$H = \frac{Vt}{2}$$

Equipment of the echo sounder

Transmission and reception are ensured by a common electro-acoustic transformer or transducer which converts the mechanical vibrations into electrical vibrations of the same frequency.

Coupled to an electric pulse generator, the transducer converts the electrical energy into acoustic energy on transmission, and conversely the reflected acoustic signal is converted into an electrical signal.

The most widely used transducers are based on the piezoelectric properties of certain ceramics (barium titanate, zirconate). They vibrate at a certain resonance frequency. These vibrations, transmitted to the water, act as sound pulses.

The optimum frequency range, which depends on the depths of water and nature of the bottom, extends from about 15 to 200 kHz, depending on the type of device. The higher the frequency, the more efficient the absorption.

At the recording end, the propagation times measured are converted into depth, depending on the speed of sound in water (from 1,460 to 1,560 m/s in sea water). For a given speed, the rate of the stylus, which inscribes along a strip of paper, determines the scale of the soundings, namely the number of metres of water represented on the width of the recording paper.

Characteristics of transducers

Transducers are characterized by their nominal frequency, directivity and level of energy.

The nominal frequency of a transducer designates its transmission frequency under permanent excitation (i.e., resonance).

For precision echo sounders, used for bathymetry, the sound beam is relatively narrow. The following are typical orders of magnitude:

- for common echo sounders:

$$10-20^{\circ}$$
 at $50-30$ kHz

- for large diameter echo sounders with very narrow beams, used at great water depths:

$$3-6^{\circ}$$
 at 30-15 kHz

The transmission level of a transducer is a measure of the energy transmitted along the axis of the transducer, measured one metre away. A high transmission for the same electric power is the sign of better efficiency.

Resolving power of an echo sounder

Resolving power of an echo sounder essentially depends on the duration of the pulse, the angle of the ultrasonic beam, the depth of the water and topography of the bottom.

A resolving power is limited by the fact that it is impossible to transmit an extremely brief signal.

If Δt is the shortest discernible time interval between two echoes, then the depth resolutions is:

$$\Delta H = \frac{V}{2} \cdot \Delta t$$

where: V is the speed of sound in water.

Principle of the side-scan sonar. Formation of the echoes

The side-scan sonar transducer acts both as transmitter and receiver of the ultrasonic signals.

The system generally consists of:

- a round-nosed cylindrical body towed from the vessel (known as the "fish"), containing one or two (1) transducers (together with the associated electronic circuits),
- a towing cable ensuring the electrical and mechanical links to the towing vessel,
- a one or two rack recorder using either electrosensitive paper or a magnetic tape.

The side-scan sonar transducer:

- transmits short sound pulses to the water, perpendicular to the direction of travel,
- receives the echoes recorded aboard the vessel (following conversion into electric pulses).

The frequencies used vary from a few tens to about 100 kHz, depending on the particular unit.

Formation of the images

The sound pulses transmitted at regular time intervals (the repetition rate essentially depends on the lateral range selected) and the echoes resulting from the irregularities on the sea bottom are recorded as a function of time (two-way trip): clearly, the nearest echoes arrive first, followed by echoes from more distant zones at ever increasing intervals.

Each group of echoes resulting from a transmission is displayed on the recorder in the form of a trace inscribed cross-wise by the stylus on the recording paper which moves longitudinally.

As the vessel advances and the pulses occur one after the other, an image is formed on the recording paper by

⁽¹⁾ The sonar is generally bilateral.

juxtapostion of the traces (somewhat similar to that obtained on a television screen).

Geometry of the ultrasonic beam

The fineness and precision of the recording are a function of the narrowness of the ultrasonic beam, and of the frequency and duration of the pulse transmitted.

The shape of the transducer is selected so as to transmit a fan-shaped beam:

- with an angle of a few degrees in the horizontal plane (azimuth),
- with an angle of about 10 to few tens of degrees in the vertical plane (elevation).

The ultrasonic beam can be broken down into the following:

- a primary lobe with an angle defined conventionally as the sector in which the sound intensity is only 3 dB beneath that of the axial (maximum) intensity,
- a number of secondary lobes.

Even though only the primary lobe is actually used in practice, the secondary lobes present a certain interest. In particular, the sub-vertical lobe:

- gives a section of the bottom of the sea along the path of the vessel,
- enables any echo from an object situated in the water near the vertical of the vessel to be identified (for instance a shoal of fish).

Formation of the echoes. Angle of incidence

The features of the bottom brought to light are:

- either of topographical nature (variation of the angle of incidence),
- or related to the physical characteristics of the soil (variations in the coefficient of reflection or backscattering).

The way in which topographic echoes are formed is shown in Fig. . All the folds in the bottom cause

the angle of incidence of the acoustic rays to vary and hence also the amount of reflected energy.

The useful part of the recording is that corresponding to angles of incidence of less than 30°, where the coefficient of reflection varies sharply with the angle of incidence. The ideal conditions therefore prevail for detecting variations in the angle of incidence and hence variations in the topography.

A change in the nature of the bottom modifies the intensity of the signal as much or even more than a change in the gradient (especially if the angle of incidence is between 20 and 60°). The reflection coefficient varies considerably when changing from mud to pebbles or rock, while sand lies somewhere in between.

Characteristics of the side-scan sonar

The side-scan sonar is essentially characterized by its longitudinal and transverse resolving powers.

Lateral range

The maximum range of a side-scan sonar depends on many factors, the leading ones being:

- the characteristics of the instrument:
- the pulse duration,
- the transmission power,
- the signal/noise ratio,
 the frequency (rF² = 1,300 is an empirical formula expressing the range in kilometres for an optimum frequency in kilocycles),
- the physico-chemical properties of the medium through which the sound waves are propagated,
- the implementation parametersthe height of the "fish" above the bottom,
- the inclination of the axis of the beam from the horizontal.

Distortion of side-scan sonar images

There are various causes for the distortion of sidescan images, including the following:

- the obliqueness of the beams
- the slope of the bottom,
- the anisotropy of the medium through which the rays propagate,
- the navigating conditions
- the scales on the recordings.

APPENDIX V-1

SUBMARINE SLUMPING AND THE INITIATION OF TURBIDITY CURRENTS

by

N. R. Morgenstern

SUBMARINE SLUMPING AND THE INITIATION OF TURBIDITY CURRENTS

ABSTRACT

The conditions under which submarine slumping is known to have occurred are reviewed and the agencies causing them are discussed. Special attention is given to earthquake effects. It is pointed out that slumps can result in a wide variety of sedimentary structures and many of these structures are associated with liquefaction. The strength of sediments is considered, and the influence of underconsolidation due to high rates of sedimentation on the strength of marine sediments is treated in detail. The mechanics of slumping are analyzed from the point of view of both drained and undrained failure. It is thought that some slopes transform into high-density turbidity currents. The evidence for the existence of such currents is summarized and a theory presented to show that a slump can achieve sufficiently high velocities to transform into a turbidity current if the pore pressures induced at failure are high enough.

INTRODUCTION

Much of the progress in understanding the processes involved in subaerial landslides has been possible only through detailed analysis of partivular cases. A minimum requirement for carrying out such an analysis is, knowledge of the slope profile, the shape and location of the major slip surface, the water pressure conditions at the time of fullure, the appropriate soil strength parameters, and the soil densities. With these data it is possible to perform faily reliable calculations to account for the movements of the soil mass. In the case of subaqueous landslides or slumps the necessary information is seldom available and few properly documented case records exist. It is therefore necessary to extrapolate from experience gained in the study of subaerial movements. It is also essential to study the fossil structures of slumps preserved in the geological record in order to establish

the conditions under which slumping has occurred and to observe the influence of the movements on the structure of the sediments. Observations of stable submarine slopes and knowledge of the properties of the sediments composing them can be used to bound the occurrence of slumps. A review of some of the information that is available regarding submarine slumping suggests that there are two problems associated with the phenomenon that deserve particular attention. The first is whether it is possible for slumps to occur on gentle slopes, particularly on the open continental shelf and slope. The second problem is to account for the wide variety of sedimentary structures that have been attributed to slumping. These range from large sheets of strata that have been transported intact to turbidites (Dzulynski and Walton, 1965). Turbidite deposits are widespread (see Boums, 1962) and their origin is still a matter of some debate. One mechanism that has been suggested is the transformation of a slump into a turbidity current and subsequent deposition of the turbidite.

Most sediments involved in slumps are likely to be normally consolidated.

However, in regions of high rates of sedimentation such as exist in some deltas, there will be a lag between the accumulation of the material and the consolidation associated with it. This gives rise to an excess pore pressure and the sediment is accordingly weaker. This underconsolidated material is evidently prone to slumping. Overconsolidated sediments also exist in a marine environment, the overconsolidation having been induced by removal of overburden by erosion of sediment during the development of submarine canyons and channels associated with sea fans. It will be seen that some very steep slopes that have been observed must be composed of material that is either overconsolidated or cemented. Nevertheless, the amount of exposure of overconsolidated material (excepting in submarine canyons) is probably small, and the influence of this aspect of sediment behavior will not be considered in any detail.

In the following, data regarding slope angles for both stable and unstable profiles are presented, and the agencies that can induce slumping are discussed. A further section reviews the various sedimentary structures that slumping can produce and shows that sediments after slumping can achieve a broad range of mobility from rigid block motion to turbulent flow. Shear strength properties of sediments are then discussed with special reference to the influence of metastability and underconsolidation. The mechanics of various modes of failure are introduced. Finally the acceleration of a soil mass moving down a slope is analyzed, and some conditions that must be satisfied for transformation into a turbidity current are suggested.

OCCURRENCE OF SLUMPING

Slumping has been observed or has been inferred to have occurred on a wide range of slope inclinations. One of the first papers to draw attention to the possibility of slumping on slopes of gentle gradient was by Heim (1908) who described the slip that flowed into Lake Zug, Switzerland, in 1887. The slope had an inclination of 2.5 degrees. Unfortunately, the reasons for the initiation of the movement are not clear. The observations of Archanguelsky (1930) are also often cited in this context. In studying a sequence of cores from the Black

Sea, he observed that recent sediments were often absent from the slope leading from the upper part of the shore terrace to the deep basin of the sea. He did, however, find such sediments in a state of intense deformation and with duplicate succession on the steeper lower slopes and concluded that they had slumped from above on inclinations of 1 to 3 degrees. Slumping on inclinations of 1 degree has been suggested by Shepard (1955) to account for the delta-front valleys associated with the Mississippi River. The existence of underconsolidated material in this region suggests that this explanation is likely. Submarine slumping of Norian strata in New Zealand has been discussed by Grant-Mackie and Lowry (1964) who describe an exposure of 530 ft of highly disturbed sediment. This layer lies within a sequence of regular undisturbed Upper Triassic strata but displays slump balls, welded contacts, and other features associated with submarine slumps. By correlating sediments and fauna the authors infer that the slope at the time of movement may have been less than 1/1 degree. Movement occurred during a period of tilting of 8 degrees by the sea floor and the slope angle quoted must be considered to be a minimum.

It should be noted that the possibility of slumping on such gentle slopes has been questioned by Moore (1961) excepting areas of rapid accumulation. In particular, Moore doubts the existence of slumping on the deep sea floor and normal open continental shelf. Regarding the continental slope, he observes that the amount of slumping will vary with the type of sediment, its rate of accumulation and the topographic features in the regions in which it is being deposited. Detailed discussion of some of Moore's conclusions will be given in a further section. However, it is of interest here to introduce some aspects of submarine topography in order to distinguish between the various gradients associated with ocean bottom features. A detailed discussion of submarine topography may be found in Shepard (1963), Hill (1963), and Menard (1964).

Moving seaward from a continent to the ocean floor, it is in general possible to distinguish between the continental shelf, continental slope and continental rise. Though by no means uniform, the average slope of the continental shelf is only 0°07' and is slightly steeper along the inner half. For the continental slope, Shepard (1963) quotes an average inclination of 4017' for the first 6000 feet of descent. Menard (1964) states that continental slopes are about 1 to 10 km high in the Pacific and have gradients of 1 to 10 degrees. However, the continental slopes are cut by submarine canyons. These are important to the problem of slumping because of the possibility of sediment accumulating in their heads, and the channeling effect that they provide for the flow of the sediment. The slopes of submarine canyons are also usually greater than that of the continental shelf. The continental rise is generally a smooth feature connecting the continental slope to the abyssal plain. Heezen and Menard (1963) quote an average gradient for the continental rise of 300:1 with some slopes as low as 700:1 and others as steep as 50:1.# Gradients of abyssal plains range from 1000:1 to 10,000:1. Other features of interest are the sediment fans at the mouths of submarine canyons, which have their origin in slump and turbidity current deposits, and the abyssal hills which are small undulations in the floor of the abyssal regions. On the basis of slope alone, it is evident that the continental slope is much more favorable for slumping than any of the other main regions mentioned above. The heads of submarine canyons provide an extremely suitable environment for slumping because of their steeper inclination and their action as sediment traps.

The effects of submarine slumping have been observed in various geological strata in many locations. Among the many examples that could be cited are the observations of Jones (1937) on Silurian rocks in North Wales and the discussion by Beets (1946) on Miocene slumping in northern Italy. Renz, Lakeman, and van der Meulen (1955) provide evidence for extensive submarine sliding in western Venezuela during the Paleocene and Eocene. For example, the geological section near the town of Carora reveals slipped masses of strongly contorted Paleocene shales containing many Cretaceous blocks and slabs. The slump material alternates with very fine-grained Paleocene sandstones and shales which were apparently deposited in quiet deep water. The authors suggest that periods of quiet sedimentation were interrupted by tectonic events along the border of

the trough. Submarine slumping on a smaller scale has been inferred by Van Straaten (1949) from the evidence of contorted glacial clays in Finland, which, he suggests, may have slid off a steep-sided esker. Finally Kuenen (1949) has described structures attributed to slumping in the Carboniferous rocks of southern Wales and he favors the view that these movements took place down slopes not exceeding a few degrees.

Subaqueous slumps on slopes inclined at steeper angles than those mentioned in an earlier paragraph have been discussed by Terzaghi (1956) and Koppejan, van Wamelen, and Weinberg (1948). These include the slope failure in clean sands and gravel in Howe Sound, British Columbia, which probably had an inclination greater than 28 degrees, and the slides composed of fine sand that occur along the coast of Zeeland. Original angles of 15 degrees are known to exist in the latter case.

Dill (1964a, 1964b, 1966) has observed in considerable detail the movement of sediment in Scripps and La Jolla submarine canyons. Slumping in fine micaceous sand occurred on inclinations of approximately 30 degrees. Sand falls over steeper inclinations and gravity creep were also important processes aiding the transport of the material down the slope.

There are many mechanisms that can induce slumping. The most common one is probably over-steepening of the slope. This may occur due to deposition or possibly crustal tilting associated with local tectonic movement. Erosion due to water currents or turbidity currents may cause local over-steepening leading to progressive failure. Slumping is purticularly common at the head of submarine canyons and in the vicinity of mouths of large rivers. These are both environments of rapid deposition. Heezen (1956) has observed that submarine cables near the mouth of the Magdalena River break most frequently in August and in the period of late November to early December. The breaks are probably due to turbidity currents initiated by submarine slumps. Progressive slumping or liquefaction are alternative mechanisms. These periods of frequent slumping correspond to the times when the river has just deposited its greatest sediment load. Dill (1964a) has found that the generation of gas associated with the decomposition of plant material that accumulates in a canyon head can lead to significant is unlikely to have any direct influence

^{*}In accord with soil mechanics practice canyon head can lead to significant a gradient quoted in this way is the ratio creep movements. Wave and storm action of a horizontal to a vertical distance. is unlikely to have any direct influence

on the stability of deeply submerged slopes. However, slides in shallow water may be triggered by erosion or rapid drawdown, nd the displaced sediment acting as a sudden load could induce failure on a slope in deeper water. Shepard (1951) has reported the results of Lathymetric traverses repeated for several years at the head of the submarine canyon at La Jolla, California. There was no correlation between storms and the observed mass movements which occurred on slopes of 5 to 8 degrees. An example of a slump which occurred in calm weather at the head of the Redondo Canyon has been given by Shepard and Emery (1941).

Loading due to severe earthquakes is widely accepted as an important agency causing slumps. Since some of these slumps may have transformed into turbidity furrents and have broken submarine calles on their descent, the source areas have been of particular interest and studies have been made of the topography. From these bathymetric surveys it is possible to approximate the slope inclimations prior to failure (Heezen and Ewing, 1952; Heezen and Ewing, 1955; Houtz, 1962; Ryan and Heezen, 1965). Gutenberg (1939) provides evidence for a submarine slide, caused by the Chilean earthquake of November 11, 1922, having occurred on a slope of about 6 degrees at a location 100 miles from the epicenter. A case of submarine slumping due to an earthquake has also been presented by Ambraseys (1960). The Alaska earthquake of March 27, 1964, caused many submarine slumps. The largest reported to date occurred at Valdez and contained an estimated volume of 75,000,000 cu m (Coulter and Migliaccio, 1966). An inclination of 6 degrees was typical of large areas of the slump, which was composed mainly of loose to medium-dense gravelly sand containing thin lenses of silt. It is of considerable interest to note that no slump toe was discovered by the post-earthquake survey, and it therefore appears that a turbidity current was formed and the sediment moved out a considerable distance from shore. There is also a history in the Valdez area of numerous cable breaks occurring during or immediately after earthquakes.

Slope inclinations in the cases mentioned above are presented in Table 1, and where the submarine slope failure lay within the epicentral region, a comment is made accordingly. The magnitude and focal depths of the shocks are also given.

The largest recorded slump occurred

in Sagami Wan, Japan, and was caused by the Kwanto earthquake of 1923. The average deepening over the area of the main slump was 100 m, and in all 7×10^{10} cu m of sediment were transported from the bay. Menard (1964) has compiled the approximate volumes of some major submarine slumps and these data are reproduced in Table 2, together with the Valdez case.

Stable slopes of various inclinations have also been observed. Kuenen (1950) reports that irrefutable evidence of slumping was not found in the deep basins of the Moluccas even though the slopes are as steep as 10 degrees in places and it is an area of high seismicity. Sea muds in thicknesses of half a meter or more have been found on slopes of at least 15 degrees. Moore (1960) has also observed recent sediments of at least one meter thickness on slopes up to 18 degrees. Buffington (1961) has found both Pleistocene sediments standing vertically and medium sand to be stable at 35 degrees in shallow water environments. During bathyscaph descents to water depths of about 3000 ft in the La Jolla fan valley, nearly horizontal beds of stiff cohesive clays alternating with cohesionless silts were found exposed in the wall of the channel, which sloped at 40 to 45 degrees (Moore, 1965). Lesser slopes in silty clay were also found. It is suggested that these steep slopes are the result of lateral erosion by turbidity currents. Slide action from the wall of the channel is also a contributing factor and explains the existence of down-slope grooves along the wall. There is no doubt that these sediments are overconsolidated. However, the ease with which the silts are disturbed suggests that diagenetic bonding may not in this case be a contributing factor to the strength of the sediments. The studies made by Emery and Terry (1956) of a submarine slope off southern California are also of interest here. Their echo-sounder profiles revealed that the shelf had an inclination of 1 degree, and the gradients of the upper portion of the slope were generally between 9 and 18 degrees. The lower slope was more regular and had an average inclination of 12 degrees. This average value is the same as that for the gullies found incising the upper slope. These gullies may be due to slumping. The slope is underlain by thick sediments, and coring with penetrations of 10 to 18 ft recovered samples of green mud. The

TABLE 1.

SOME SLUMPS CAUSED BY EARTHQUAKES

| Location and Date | Slope degrees | Magnitude M | Foénl Depth km | Within Epicentral Region | Reference |
|-------------------------------------|------------------|----------------|----------------------|--------------------------------|---|
| Grand Banks, 1929 | 3.5 | 7.2 | Shallow | Yes | Heezen and Ewing (1952) |
| Orleansville, 1954 | 4-20 | 6.7 | 7 . | No | Heezen and Ewing (1955) |
| Strait of Messina, 1908 | L, | 7.5 | 8 | Yes | Ryan and Heezen (1965) |
| Suva, 1953 | 3 | 6.75 | 60 | Yes | Houtz (1962) |
| Chile, 1922 | 6 | 8.3 | Shallow | No | Gutenberg (1939) |
| Valdez, 1964 | 6 | 8.5 | Shallow | y Yes | Coulter and Migliaccio (1966) |
| Aegean Archipelago, July 9, 1956 | 10 | 7.5 | 15 | No | Ambraseys (1960) and Admiralty Chart No. 1866 (1951), Royal Hellenic Navy |

TABLE 2. VOLUMES OF SUBMARINE SLUMPS

| Location | Volume m ³ | | |
|-------------------------|--------------------------|--|--|
| Magdalena River Delta | 3 x 10 ⁸ | | |
| Mississippi River Delta | 4 × 10 ⁷ | | |
| Suva, Fiji | 1.5 × 10 ⁸ | | |
| Valdez, Alaska | 7.5 x 10 ⁷ | | |
| Folla Fjord | 3 x 10 ⁵ | | |
| Orkdals Fjord | 107 | | |
| Sagami Wan | 7×10^{10} | | |

grain size of the specimens seaward of the self break decreases with depth in an orderly way which suggests continuous deposition. The authors provide some cross sections with soil mechanics classification data. Of considerable importance are the quantitative data that a marine sediment 5 ft below the mud-line having a liquid limit of 55 percent, a plastic limit of 30 percent, and a natural moisture content of 70 percent is presently stable on a slope of approximately 15 degrees in an area of considerable seismic activity.

SEDIMENTARY STRUCTURES ASSOCIATED WITH SLUMPING

It is beyond the scope of this study to discuss in detail the many sedimentary structures whose origin has been associated with submarine slumps and the mass movements that ensue from them. However, it is of interest to review briefly the wide variety of slump structures that have been observed, because of the information this provides for assessing the problem of the mobility of sediments after movement has begun. More comprehensive studies have been provided by Bouma (1962), Dott (1963), and Dzulynski and Walton (1965).

It is possible to distinguish four major divisions of increasing mobility of moving sediment. This is not to imply that any slump must pass through each division, but it is simply a classification to illustrate the decreasing disorder of initial sedimentary structure. The first stage is a coherent slump where little mixing of sediment has occurred and the beds have retained their identity to a large degree. Features associated with this type of slump are pull-apart structures with intrusion of sandstone dikes as described by Kuenen the pore pressures would dissipate (1953) and intraformational folding as described by Fairbridge (1946). The distinguishing feature of this division is that either the beds have not moved very far or the composition of the sediment above the slip surface gave it sufficient shearing resistance to maintain coherence even though it was intensely deformed. The second stage, which Dzulynski (1963) has called an incoherent slump, occurs when there has been extensive mixing of indurated sediment in a mass of sand, silt, or clay. Examples for this division are the slump structures mapped in Venezuela (Renz, Lakeman, and van der Heulen, 1955) and

the features in flysch described by Dzulynski and Slaczka (1958) where the section contains many slump balls. origin of pebbly mudstones (Crowell, 1957) is also probably due to incoherent slumping. The third division in increasing mobility results in fluxoturbidites. Here the mixing of the sediment and its velocity are not sufficient to develop the features characteristic of turbidites, which are the structures resulting from the final division, that is, turbidity currents. Graded bedding is an important criterion for distinguishing turbidites. It is possible that some turbidite structures can be explained by the pulsating bottom currents observed by Dill (1966).

Liquefaction plays an important role in causing many minor features observed in slumps, as well as decreasing the overall shearing resistance of the sediment and hence increasing its mobility. Liquefaction occurs most commonly in saturated loose sands and silts which, when loaded, collapse and transfer the load to the pore water. Pore pressure gradients can be set up which eliminate the shearing resistance of the sediment, and if the seepage velocity due to the hydraulic gradient is high enough, solid particles can be carried with the flow. Liquefaction is the cause of the sandstone dikes mentioned in the previous paragraph and the extensive sand volcanoes described by Gill and Kuenen (1957). In the latter case, the field evidence has prompted the authors to note that the extrusion of the sediment required a considerable period of time, starting in some cases before movement had ceased and in others after planing off of the slumped masses.

Terzaghi (1956) argued against the existence of slump-initiated turbidity currents on the basis of the short duration of liquefaction. He felt that quickly and that the slump material would come to rest within a relatively short distance from its original location. However, after the Alaska Good Friday earthquake, sandspouting occurred for a duration of 5 to 10 minutes and it is likely that excess pore pressures existed within the sediment for longer than that (Reimnitz and Marshall, 1965). It is also common experience that sediments that have been liquefied after an earthquake remain extremely soft for some time. A more detailed discussion of the influence of pore-pressure dissipation on velocity of slump movements will be given in a later section.

Terzaghi and Peck (1948) state that a saturated sand must have a relative density less than 0.4 or 0.5 before it can start to flow. They also observe that the most unstable sediments have an effective size, D₁₀, less than 0.1 mm, and a uniformity coefficient,

$$\frac{D^{10}}{D^{60}}$$

less than 5. It is of interest to analyze the gradings of some slump and turbidity current deposits to see if they meet this criterion. This only provides a necessary condition that these materials were prone to liquefaction. It is possible that part of the initial grading was deposited elsewhere and the data being compared are not representative. The effective sizes and uniformity coefficients are given in Table 3 and for comparative purposes results from sediments liquefied after the Niigata earthquake of 1960 (Kishida, 1965) and from a fine sand which almost liquefied during laboratory shear tests (Bjerrum, Kringstad, and Kummeneje, 1961) are included.

Each case quoted in Table 3 including the complete graded sea bed from the Hudson sea fan, satisfies the criterion put forward by Terzaghi and Peck. Although this alone by no means establishes liquefaction as a mechanism, at least the grading of these deposits suggests that the source sediments may be prone to it.

STRENGTH OF SEDIMENTS

In terms of effective stress, the shear resistance along a plane of failure in a saturated soil is given by

$$\tau_f = c' + (\sigma - u) \tan \phi' \tag{1}$$

where τ_f denotes the shear stress on

- o' denotes the angle of \ fective
 shearing resistance \ stress
- denotes the total stress normal to the failure plane

and u denotes the pore pressure.

TABLE 3.

EFFECTIVE SIZES AND UNIFORMITY COEFFICIENTS

| Sedi | ment | Effective Size D10 (mm) | Uniformity Coefficient DGO D10 | Reference |
|---------------------------|------------------------------|-------------------------|---|---|
| Core A180-1, | . Top | .016 | 3.3 | Heezen (1963) |
| Core A180-2 | , 64 cm | .016 | 3.8 | " |
| Hudson Sea 1 | Fan 0-4 cm | .055 | 4.4 | Kuenen (1964) |
| ** | 4-18 cm | .035 | 3.7 | 11 |
| • | 18-24 cm | .053 | 3.0 | u |
| п | 24-48 cm | .053 | 3. կ | H |
| 11 | 48-72 cm | 060 | 3.3 | u u |
| San Pedro B portion of | asin (lower graded layer) | . 062 | 2.6 | Gorsline and Emery (1959) |
| Niigata | | . 09 | 2.8 | Kishida (1965) |
| Fine Sand | | •07 | 2.5 | Bjerrum, Kringstad, and Kummeneje (1961) |

His experiments were carried out under isotropic consolidation and this will in general result in a higher value of the

ratio (Skempton and Bishop, 1954). The actual difference is difficult to estimate because the pore pressure parameter, $A_{\rm f}$, depends upon the history of consolidation. It is likely that the most dominant factor accounting for the deviation from the correlation is carbonate bonding. Assuming the relation of Figure 2 to hold, a predicted value of

can be obtained from the plasticity index data given by Moore. Figure 3 shows that the ratio of the predicted to measured values decreases with increasing carbonate content. Higher values of

$$\frac{c}{D}$$

than might be expected have also been found in short cores of shallow water sediments from Lower Chesapeake Bay (Horrison, Lynch, and Altschaeffl, 1964) and in short cores of deep-sea sediments (Richards, 1962). Fisk and Mc-Clelland (1959), however, report that fully consolidated sediments from the Mississippi delta agree with the correlation. Although it is premature to generalize with regard to the undrained strength of recent marine sediments, it is unlikely that a fully consolidated stable material will have an undrained strength below the relation shown in Figure 2.

Terzaghi (1956) drew attention to the influence of high rates of sedimentation on the development of strength in a consolidating sediment. Excess pore pressures can develop in a stratum that is undergoing an increase in height due to deposition. These excess pore pressures will depend upon the rate of sedimentation, the height of the stratum, and the coefficient of consolidation of the material. The excess pore pressure at any level in the stratum will reduce the effective stress under which the material has been consolidated and, as is evident from equation (3), the undrained strength at that level will be reduced accordingly.

Consider the stratum shown in Figure 4. When fully consolidated, the maximum effective overburden pressures, \mathbf{p}_{m} , at some depth, \mathbf{z}_{i} is given by

$$p_{m} = \gamma'z \qquad (6)$$

where γ' is the submerged density of the soil, assumed constant with depth. The increase of undrained strength with depth for a fully consolidated material may be denoted by

$$\frac{c}{p_m} = N \tag{7}$$

If during consolidation excess pore pressures exist as shown diagrammatically in Figure 4, the effective overburden pressure, p, at any instant is

$$p = \gamma'z - u = \gamma'z(1 - \frac{u}{\gamma'z})$$
 (8)

where u is the excess pore pressure at that instant. At any instant the excess pore pressure isochrome may be approximated by a linear variation with depth,

$$\mathbf{u} = \mathbf{n}\mathbf{z} \tag{9}$$

and equation (8) becomes

$$p = \gamma' z (1 - \frac{n}{\gamma'}) \qquad (10)$$

However,

$$1 - \frac{n}{\gamma'} = \overline{\nu} \tag{11}$$

where $\bar{\nu}$ is the average degree of consolidation. Therefore the undrained strength available in an underconsolidated clay should be proportional to the average degree of consolidation, that is,

$$\left(\frac{c}{p_m}\right)_{\bar{U}} = N\bar{v} \tag{12}$$

Estimates of the degree of consolidation in a layer subject to sedimentation at a constant rate can be obtained from the solution presented by Gibson (1958) for the problem of the progress of consolidation in a clay layer which increases in thickness with time. Considering a layer growing on an impermeable base at a constant rate, it is of interest to calculate the degree of consolidation for a range of rates of sedimentation and coefficients of consolidation when the layer has

For normally consolidated clays and granular soils, the apparent cohesion is zero and equation (1) becomes

$$\tau_f = (\sigma - u) \tan \phi'$$

It is possible to distinguish between structurally stable and structurally metastable soils. Metastable soils show a very large rate of volume decrease during drained shear and may even display an initial yield point at a stress less than their maximum strength. Some stress-strain relations for stable and metastable soils are shown diagrammatically in Figure 1.

Quick clays and very loose sands are examples of structurally metastable soils which may be defined as soils that, when brought to failure under drained conditions, deform further under under undrained conditions.

For stable clays ¢' varies between 20 and 35 degrees. A correlation between ¢' and plasticity index has been given by Bjerrum and Simons (1961). Stable loose silts and sands typically have values of ¢' between 28 and 34 degrees.

Large deformations in soils containing a clay content greater than approximately 35 per cent induce preferred orientation of the clay particles in the shear zone and cause a reduction of ¢' (Skempton, 1964). Angles of shearing resistance as low as 10 degrees are not uncommon in clays that have been subject to large strains. Tew data giving strength parameters in terms of effective stress are available for present day marine sediments. The results of Moore (1961, 1962) are ambiguous because the conditions of drainage in his tests are not adequately defined. This is not the case for the strength data for sediments from the experimental Mohole (Moore, 1964). The average of six results on the calcareous silty clay from one borehole gives a ¢' of 28 degrees and a c' of about 8 psi. There is as yet no evidence to suggest that the effective stress strength paramenters of stable deep-sea deposits will be any lower than the range commonly encountered on land. Indeed, the presence of diagenetic bonding agents in some marine environents can make the sediment stronger than the usual range.

When a fully saturated soil is sheared under undrained conditions and the results are interpreted in terms of total stresses, the material behaves as though it is purely cohesive. This holds for saturated sands as well as for

clays (Bishop and Eldin, 1950). For a normally consolidated clay or a sand in the ground, the undrained shear strength, c_u, is related to the stresses under which the soil has been consolidated, the effective angle of shearing resistance, and the pore pressures at failure by:

$$c_u = \frac{p \sin \phi' \left[K + (1 - K)A_f\right]}{1 + (2A_f - 1)\sin \phi'}$$
 (3)

where p denotes the vertical effective pressure,

K denotes the ratio between the horizontal and vertical effective pressures,

and Af is the appropriate pore pressure parameter at failure (Skempton, 1954).

For stress conditions associated with no lateral yielding, as might be assumed to exist during deposition either horizontally or on a gentle inclination, K may be expressed empirically by (Bishop, 1958):

$$K = 1 - \sin \phi' \tag{4}$$

Equation (3) then becomes

$$\frac{c_u}{p} = \frac{\sin \phi' \left[1 - \sin \phi' + A_f \sin \phi'\right]}{1 + (2A_f - 1)\sin \phi'}$$
(5)

For any particular fully consolidated soil, the ratio

is a constant and indicates that the undrained strength increases with depth. It is know that this ratio correlates closely with the plasticity index of many marine clays (Skempton, 1957), and the correlation is given in Figure 2. Owing to sample disturbance and improvements in testing technique since the data were gathered, this relation may be considered to be a lower boundary to the true relation. However, there is no reason to expect that more refined data will produce major changes in the relation.

Moore (1964) has shown that the strength data from the Mohole sediments lie appreciably above the correlation. As he has observed, there are at least two factors which may account for this.

grown to a height that might be typical of a significant submarine slump. A height of 15 m has been assumed, and coefficients of consolidation from 1 \times 10^{-5} cm²/sec for a clay to 1 \times 10^{-2} cm2/sec for a coarse silt have been adopted. The degrees of consolidation of the layer for a range of rates of deposition from abyssal conditions to extreme deltaic conditions have been computed and are given in Figure 5, plotted against the rate of sedimentation for the range of consolidation parameters chosen. The results reveal that for a layer of this thickness, underconsolidation is only significant for silty clays and clays deposited at deltaic rates. Since the heads of some submarine canyons act as sediment traps, the rate of accumulation may be sufficiently high to suggest that underconsolidation is a factor associated with slumping in them. It is also possible to speculate that slumping occurred more frequently in the Pleistocene, during the recession of the glaciers, because of higher rates of sedimentation. This, together with turbidity current erosion and a lowered sea level during the Pleistocene, may be the dominant mechanism accounting for the origin of many submarine canyons (Kuenen, 1950; Shepard, 1983).

Subject to some assumptions, the relation between underconsolidation and strength presented in equation (12) is corroborated by the observations of Fisk and McClelland (1959) on the deltaic deposits on the continental shelf off Louisiana. The authors provide data for three locations of similar composition, but of different degrees of consolidation and hence of different strengths. The relevant information is assembled in Table 4.

Evidence of full consolidation for the Eugene Island stratum is provided by the fit of the

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and plasticity index values with the correlation in Figure 2. For purposes of comparison the three cases are plotted on Figure 2. Assuming that the 96 ft of the Eugene Island sediment were deposited in 10,000 years gives a rate of sedimentation of 0.29 cm per year. Theoretically, infinite time is required for full consolidation. However, if it is assumed that consolidation is essentially complete when the degree of consolidation is 95 percent, it is possible

to compute the coefficient of consolidation for the material from the theoretical relation obtained by Gibson (1958). A value of 2.7 \times 10⁻⁴ cm² per sec is found, which is quite reasonable, considering the Atterberg limits of the material. Now, using this value, it is possible to compute the average degree of consolidation for the two other locations if the rates of sedimentation can be fixed. For the Grand Isle location, a rate of sedimentation of 3.5 cm per year has been used, based upon the accumulation of 170 ft in 1500 years. In the case of the South Pass location the base of the layer is indistinct, but bounds for its thickness have been given. Calculations have been carried out for both bounds with a time for deposition of 450 years. The computed degreés of consolidation are given in Table 5, together with the ratio of the observed

> c_u P

value to the maximum. The relation between degree of consolidation and available strength for this sediment is plotted in Figure 6, and it is seen that the linear relationship of equation (12) fits the data extremely well.

Metastable sands and silts which are prone to liquefaction are difficult to obtain in an undisturbed state. They are also difficult to reproduce in the laboratory, and therefore reliable data concerning their behavior are accordingly rare. Bjerrum, Kringstad, and Kummeneje (1961), however, have succeeded in carrying out both drained and undrained triaxial compression tests on a very loose fine sand. Their observations of the low strength mobilized are of particular interest. Under fully drained conditions, values of \$\phi'\$ as low as 19 degrees were found. Under undrained conditions, the very loose sand showed values of &' as low as 11 degrees and a ratio of undrained strength to effective consolidation pressure as low as 0.11. The pore pressures set up during undrained failure were very high. Values of A of 2.7 were observed at failure and the results of one typical test showed that A continued to increase after failure to approximately 9. It is evident that both the drained and undrained strengths of very loose sands are much lower than those of corresponding stable materials. The undrained strengths are comparable to the lowest values observed in normally consolidated marine clays. Further-

TABLE 4.

DELTAIC DEPOSITS OFF LOUISIANA (FISK AND McCLELLAND, 1959)

| Location | State | Plasticity Liquid Plastic Index c | | | | Banks | |
|---|-----------------------------|-----------------------------------|---------|--------------------|-------------------|------------------|-------------------------|
| Document of the second of the | State | • | Limit % | Index 7 (average) | e u | Depth ft | Age Years |
| Eugene Island Block 188 | Fully consoli- | 80- 90 | 25-30 | 53 | 0.31 | 96 | not less than 10,000 |
| Grand Isle Block 23 | Underconsoli- dated | 80-90 | 25-30 | 53 . | 0.15 | 170 | not more than 1500 |
| South Pass Block 20 | Very undercon- solidated | 60-100 | 20-30 | 55 | 0.028 (average | 255 -3 20 | 450 |

TABLE 5.

UNDERCONSOLIDATION OF DELTAIC DEPOSITS OFF LOUISIANA

| Location | Rate of | Sedimentation cm/year | Average Degree of Consolidation | cu (observed) cu (maximum) |
|--------------------------|---------|-----------------------|------------------------------------|----------------------------|
| Eugene Isla Block 188 | nd | 0.29 | 1.00 | 1.00 |
| Grand Isle Block 23 | | 3.5 | 0.48 | 0.48 |
| South Pass Block 20 | | 17 21.6 | 0.08 | 0.09 |

more, the exceedingly high pore.pressures set up during undrained failure are probably an important factor aiding the post-failure mobility of such metastable materials.

Seed and Lee (1964) have studied the influence on the strength of a fine silty sand of pulsating loads such as might occur during an earthquake, and they demonstrated that in a given material consolidated to a particular void ratio, the deviator stress required to cause failure decreases with the number of pulses to failure. This also depends upon the principal stress ratio during consolidation and the manner in which the pulsating load test is carried out. Seed and Lee have found

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values less than 0.1 for loose cohesionless soils subject to pulsating load.

Observations on the strength of sensitive clays, such as the quick clays of Scandinavia, may also have a bearing on the possible in-place strength of cohesive submarine sediments, if, due to the formation of weak bonds, they develop a loose structure. Bjerrum (1961) has discussed in detail the strength of materials with loose structure, and he cites tests on quick clay which gave drained angles of shearing resistance between 9 and 13 degrees. Of particular importance here is the

observation that in undrained tests on such material, failure may occur before the trictional resistance is fully mobilized.

MECHANICS OF SLUMPING

As Moore (1961) has indicated, consideration of the equilibrium of an infinite slope with failure occurring on a plane or planes parallel to the slope provides an adequate framework within which to discuss the mechanics of slumping. It is possible to consider more complicated configurations (for example, Morgenstern and Price, 1965); however, the available data regarding slope profiles, sediment strength, and initiating mechanism are insufficient to warrant this. The strength of any sediment depends, among other things, upon the conditions of drainage operating during shear. It is therefore essential to distinguish between drained and undrained slumping. It will be seen that the slope inclination at which slumping occurs is strongly dependent upon whether the initiuting process induces a drained or an undrained slump. A third type of slumping, termed collapse slumping, may also be denoted. This type of slumping is associated with metastable sediments, and although it has only been studied in a subscript environment, the possibility of formation of metastable sediments in a marine environment suggests that collapse slumping may be an important mechanism there. It will be defined and discussed in more detail in a later paragraph.

No excess pore pressures exist at failure in a drained slump. By considering the horizontal and vertical equilibrium of a slice shown in Figure 7, the relation between the slope angle at failure and the properties of the scdiment may be readily shown to be

$$\tan \alpha = \tan \phi^{\dagger} + \frac{c^{\dagger}}{\gamma^{\dagger}h} \times \sec^2 \alpha \qquad (13)$$

where a denotes the inclination of the slope to the horizontal

4' denotes the angle of in terms shearing resistance of efc' denotes the apparent fective cohesion stress
y'denotes submerged density of the sediment

and h denotes the height of sediment participating in the slump. It is of interest to note that a comparable analysis for subaerial condi-

tions would involve the bulk density of the material in the resulting form of equation (13). Therefore a given amount of cohesion is more effective in maintaining stability under submarine conditions, all other conditions being the came. When the sediment is a normally consolidated clay or an uncemented sand or silt, the following well-known relation holds at failure:

 $tan \alpha = tan \phi'$ (14)

Drained slumping is most commonly caused by depositional oversteepening. Since the o' for stable material is generally greater than 20 degrees, and few features in deep water have inclinations as steep as this, it appears that drained slumping of stable sediments is not a dominant mechanism. It can, however, occur on the steep slopes of erosion channels. Steep slopes such as those observed by Moore (1965) require the existence of some cohesion whose origin is either in overconsolidation or cementing to account for their stability. Terzaghi (1956) stated that steep slopes of coarsegrained sediments are most commonly encountered in deltas deposited by mountain streams and cited the sand and gravel delta of Howe Sound, British Columbia, as an example. Here slope angles of 27 to 28 degrees are stable. The slump which occurred here must have originally had a slope steeper than this, and Terzaghi suggested that residual pore pressures after drawlown reduced the shearing resistance sufficiently to cause failure. This is not a drained slump like those considered above. The influence of drawdown pore pressures may be estimated by methods commonly used in the design of earth dams (Bishop, 1957; Bishop and Morgenstern, 1960) and will not be considered further here. Under fully drained conditions the mobility of the sediment will be small and it will come to rest when the slope angle is slightly less than the angle of shearing resistance. Mobility under undrained conditions will be considered in the section relating to the initiation of turbidity currents.

Undrained slumps may be caused by stresses set up during rapid deposition or erosion. Dynamic loading due to earthquakes will also produce undrained failure. Slumping in underconsolidated sediment is also best considered in terms of the undrained strength of the material.

The influence of an earthquake in the analysis of undrained slumping may be incorporated by introducing a horizontal body force, k, as some percentage of gravity and considering the equilibrium of a slice in the infinite slope. Earthquakes will in general also produce a vertical acceleration, but this is usually less than the horizontal acceleration, and for simplicity will be neglected here.

Considering the equilibrium of the slice shown in Figure 8, and resolving forces parallel to the slope one obtains

$$Cu \cdot 1 = W' \cdot \sin \alpha + k \cdot W \cdot \cos \alpha \tag{15}$$

where Cu denotes the undrained strength mobilized at failure

- W' denotes the submerged density of the slice and is given by y' · b · h
- W denotes the bulk density of the slice and is given by h. . h.
- 1 is the length along the base of the slice

and k is some percentage of gravity.

After simplification, equation (15)

reduces to

$$\frac{c_0}{\gamma^{\dagger}h} = \frac{1}{2} \sin 2\alpha + k \cdot \frac{\gamma}{\gamma^{\dagger}} \cdot \cos^2 \alpha \quad (16)$$

Equation (16) relates, for undrained slumping, the slope angle at which failure takes place to the undrained strength and density of the sediment, the height of the slope, and the horizontal earthquake acceleration, if any. For slopes of gentle inclination

$$\frac{Cu}{\gamma^{+}h} = \frac{Cu}{P} = N \tag{17}$$

and for many sediments

$$\gamma = 3\gamma' \tag{18}$$

Equation (16) now becomes

$$N = \frac{1}{2} \sin 2\alpha + 3k \cos^2 \alpha \tag{19}$$

Values of N required to equilibrate a range of slopes inclined from 0 to 20 degrees, and subject to horizontal accelerations up to 15 percent of gravity, have been computed and are plotted in Figure 9. Considering first the stability of slopes free of earthquake loading, if the observed range

of N values for most normally consolidated sediments (Figure 2) is taken to apply (N<0.4), few slopes subject to undrained loading can stand at inclinations greater than 25 degrees. Overconsolidated sediments and sediments with strong diagenetic bonds can. of course, stand more steeply. Slumping on very gentle gradients of, say, less than 2 degrees, without the aid of earthquakes, can only occur in very underconsolidated material. Terzaghi (1956) and Moore (1961) have already drawn attention to the evidence that the low strengths of the very underconsolidated Mississippi delta sediments are consistent with slumping on slope angles barely in excess of 1 degree. If very loose, cohesionless sediments have an II value of about 0.11 as found by Bjerrum, Kringstad, and Kummeneje (1961) it is seen that failure takes place on slopes of about 6 degrees, and it is of interest to note that this is a fairly typical inclination for the continental shelf.

Figure 9 shows that even small earthquake-induced accelerations are very detrimental to the stability of a submarine slope. However, in a detailed study of mass transport of sediment in the heads of Scripps Submarine Canyon, California, Chamberlain (1964) concluded that there is insufficient reason to believe that a relationship exists between the occurrence of submarine canyon deepenings and earthquake disturbances. Based on direct observations, Dill (1964a) states that earthquakes have little effect on the failures that cause the removal of sediment from the head of Scripps Canyon. The slope failures caused by earthquakes listed in Table 1 provide evidence that there is at least a correlation between submarine slumping and near earthquakes of large magnitude. It seems significant that all the shocks cited in this table had a magnitude greater than 6.5. Taking 6 degrees as a typical angle representing some of the cases listed in Table 1, and assuming the sediment to have undrained strengths in terms of N between .25 and .40. it is seen from Figure 9 that the slope must have responded with an acceleration between 5 and 10 percent of gravity.

The observations of Emery and Terry (1956), described in an earlier section, provide an interesting case of a relatively steep stable slope in a seismically active area. Since the sediment has a plasticity index of about

25 percent, the value of N might, from Figure 2, be at least 0.22 and the equilibrium slope for undrained failure without earthquake loading is 13 degrees. This fits well within the range of the observed slope angles and is close to the average of 12 degrees. However, steeper slopes were observed, and the index data quoted above refer to a slope of approximately 15 degrees. A slope of 15 degrees requires an N value of 0.25 for stability. This is within the scatter to be expected from correlation with Figure 2, but it leaves no margin for incorporating the influence of earthquake loading. To obviate this difficulty, it is worthwhile noting that although bedrock accelerations during an earthquake may be high, the response of the overlying sediment depends upon its modulus of rigidity, and if this is very low, the shear stresses induced in the sediment may be low, although the displacements will be large.* In a normally consolidated sediment the modulus of rigidity will vary with depth, and it could be that for typical ground motions associated with near earthquakes of magnitude less than 6, the dynamic stresses in the sediment are not very significant. data on the variation of rigidity with derth in a slope could be obtained, the solution given by Ambraseys (1959) to the problem of the response to an arbitrary ground motion of an elastic overburden with varying rigidity could be used to investigate this point.

A collapse slump is defined as one that fails initially under drained conditions, but the deformations associated with failure bring about a large increase in pore pressures. These pore pressures reduce the shearing resistance, and the soil mass accelerates. This

*The dynamic shear stress in the sediment is given by:

$$\tau_{\mathbf{d}} = \frac{\mathbf{Y}}{\mathbf{g}} \cdot \mathbf{V} \mathbf{s} \cdot \dot{\mathbf{u}} \tag{20}$$

where τ_d denotes the dynamic shear stress Vs denotes the shear wave velocity $\mathring{\mathbf{u}}$ denotes the particle velocity

and $\frac{Y}{\sigma}$ denotes the mass density.

If the computed response of the sediment to earthquake loading shows low strain rates and hence low particle velocities, and if Vs is small due to the low rigidity, the dynamic stress, τ_d , will also be small.

type of mechanism has only received detailed attention in the study of one landslide which occurred in a thin layer of quick clay (Hutchinson, 1961). It is probably a feature peculiar to structurally metastable sediments. analysis of this slide, using pore pressures based upon ground water level observations, indicated that failure occurred with a drained angle of shearing resistance of only 7 ± 1.5 degrees. This value was substantiated by both in-place and laboratory shear box tests. Conventional isotropically consolidated undrained triaxial tests gave values of of 25 degrees, and Bjerrum (1961) has suggested that the lower initial yield is destroyed by sample disturbance and reconsolidation. Further information on this phenomenon is given by Bjerrum and Landva (1966). Hutchinson (1961) also observed pore pressures in excess of hydrostatic pressure within the clay layer and remarked that the sliding caused breakdown of the clay structure, and hence part of the overburden load was transferred to the pore water. Therefore, although the initial failure occurred under drained conditions, further movement occurred under undrained conditions. This can only happen when the undrained resistance is less than the drained resistance at failure, as it was in the case discussed

Although these quick clays do not commonly exist in a submarine environment because they have been made metastable by the leaching of salt water, some submarine sediments may achieve metastability and high sensitivity in other ways and could be subject to collapse slumping. Therefore the possibility of initial slumping under drained conditions with acceleration under undrained conditions on slopes of 5 to 10 degrees cannot be excluded without further study.

Moore (1961) concluded that in general most sediments are theoretically stable to great thicknesses on very steep slopes. This conclusion was based upon the use of strength parameters typical for drained compression of stable sediments, and the analysis presented here, for this case, is in agreement. Undrained failure of stable, fully consolidated sediments can lead to slumping on slopes of more gentle inclination, particularly if the sediment responds to earthquake loading with a significant acceleration. Therefore considerable slumping may occur on the normal open shelf where

collapse slumping may also be important. In agreement with Moore, the deep sea is probably almost free of slumping. This is because the gradients of most physical features there are very low; sediments are likely to be fully consolidated and possibly stronger due to diagenetic bonding, and the slopes are situated out of range of several of the agencies which can produce undrained failure. Slumping is undoubtedly frequent in areas of rapid deposition, and here may occur on very gentle gradients.

INITIATION OF TURBIDITY CURRENTS

When a slump takes place in a stable cohesive sediment of low sensitivity, experience of subaerial landslides suggests that shearing will take place on a plane or set of planes while the mass of the sediment remains relatively intact. The mass of sediment should come to rest at a new equilibrium position consistent with the strength obtaining after failure, and although it may exhibit features associated with a coherent slump, such as introformational folding, it is difficult to imagine that the stresses acting on the slump mass during motion can disrupt its structure sufficiently to allow dispersion of the sediment and mixing with water. However, cohesive sediments of high sensitivity and cohesionless soils, particularly metastable ones, can achieve a greater mobility, and in the limit a slump may he transformed into a turbidity current.

There is considerable evidence that some sediments in the deep sea have had their origin in shallow water. In a study of deep-sea sands, Kuenen (1964) stated that practically all deep-sea sands were emplaced by turbidity currents. Heezen and Hollister (1964) suggested that although deep-sea currents are capable of transporting coarse material, they cannot account for the graded bedding which is a common feature of deep-sea sands. However, in the light of Dill's observations (1964a, 1966) of bottom current pulsations and creep and slump effects. these conclusions are possibly premature, and the presence of deep-sea sands cannot be taken as wholly unambiguous evidence for the existence of turbidity currents. Other evidence for turbidity current deposition includes the displacement of shallow-water benthonic fauna to deep water, and the relief

and distribution of abyssal plains, channels, and fans (Menard, 1964). The timing of submarine cable breaks, after slumping was caused by an earthquake, demonstrates the mobility of the sediment. The first confirmation that a slump can transform into a turbidity current was given by Heezen, Ericson, and Ewing (1954), who discovered a graded bed of silt south of the Grand Banks. This bed had its origin in a turbidity current caused by the slump which occurred during the earthquake of 1929. Heezen and Drake (1964) have suggested that there was deep-seated coherent slumping as well in this case. Slumping has also been cited by Holtedahl (1965) as the initiating agency to account for the abundant recent turbidites found in the Hardangerfjord, Norway.

Not all turbidity currents have their origin in slumps. In the case of the Congo Submarine Canyons (Heezen and others, 1964) cable breaks occurred most frequently at the times of greatest bed load discharge, and since a delta is not being formed at the river mouth, it is possible that large sediment discharges continue directly as turbidity flows.

Only low density turbidity currents have been directly observed. These often occur due to the discharge of sediment by a river into a lake or reservoir. In the case of the Lake Mead turbidity current, it is known that the excess density is only about 1 percent and the velocity less than 2 ft per sec on a gradient of approximately 2000:1 (Gould, 1951). Kuenen (1950) postulated the existence of turbidity currents with densities comparable to the bulk density of typical sediments and was able to produce them in the laboratory. The density of turbidity currents in the sea remains debatable. The high-density current explains sea-floor phenomena more easily, but is yet to be observed. If the low-density current begins as a slump, it is not clear how the extreme dispersion of the sediment occurs. The twisting and abrasion of cables broken by the Suva turbidity current described by Houtz and Wellman (1962) favors the high density interpretation. Alternative mechanisms for a sequence of cable breaks, such as a wave of liquefaction or progressive slumping, appear less satisfactory.

Data on times of breakage of submarine cables provide evidence that turbidity currents can maintain velocities of about 15 to 30 ft per sec on the very gentle gradients of the abyssal plains. Although it is generally accepted that higher velocities are developed on the steeper continental slope, few conclusive data are available and the exact values are still debated. Menard (196%) suggests that the Crand Banks turbidity current reached a velocity of 63 ft per sec before it legan to decelerate, and even higher values have been quoted.

While there has been considerable study of the mechanics of turbidity flow (see Johnson, 1962, 1964, for a review) little attention has been paid to the problem of how a current is initiated. Moreover, small-scale experiments carried out on a naturally sloping sea floor 40 ft below sea level were not successful in producing a high-density, high-velocity current (Buffington, 1961). In the following, the acceleration of a slump after failure is considered in an attempt to delineate some of the conditions necessary for a slump to attain sufficient velocity that it may transform into a turbidity current. These considerations may explain the failure of the experiments mentioned previously.

The problem is best treated in terms of effective stress. It is assumed that some unspecified mechanism has brought the cohesionless sediment on an infinite slope into a state of limiting equilibrium by inducing an undrained failure, and that the excess pore pressure in the sediment at this instant is given by

$$u = nz \tag{21}$$

where u denotes the excess pore pressure

n is some number

and z is measured perpendicular to the slope, increasing downwards from the surface of the slope. If the slice shown in Figure 10 is to be in a state of limiting equilibrium, it is readily shown that

$$\frac{n}{\gamma} = \frac{\cos \alpha \tan \phi^{\dagger} - \sin \alpha}{\tan \phi^{\dagger}}$$
 (22)

From equation (22) the values of $\frac{n}{\gamma}$ have been computed for a range of slope angles and for values of ϕ ' of 10, 20, and 30 degrees. These values are plotted in Figure 11. If for a given value of α and ϕ ' the magnitude of

obtaining in the slope is less than that shown in Figure 11, motion will not occur. If, however, it is greater, though not necessarily liquefied, the sediment will not be in equilibrium and it will accelerate due to the force unbalance acting upon the mass. (The viscous stress acting on the upper surface may be neglected.) Assuming that the mass is initially at rest, the equation of motion gives

 $V_r = \frac{g}{\gamma} [\gamma' \sin -(\gamma' \cos \alpha - n) \tan \phi'] t$ (23)

where V_r denotes velocity for this rigid block model

t denotes time

and g denotes the acceleration due to gravity.

It is seen that for this model the velocity increases linearly with time, and depends upon the slope angle, the excess pore pressure gradient, and the density and strength of the sediment. A diagrammatic velocity profile is shown in Figure 10.

A more realistic model may be developed by incorporating a viscous resistance due to the strain rate in the sediment. This would give rise to a velocity profile of the type shown for this mode of flow in Figure 10. Since the slope is infinite there is no variation of any stress or strain-rate in the x direction. The equation of motion for an infinitesimal element accelerating in the x direction becomes

$$\gamma' \sin \alpha - \frac{\partial \tau_{xz}}{\partial z} = \frac{\gamma}{g} \frac{\partial V_{y}}{\partial \tau}$$
 (24)

where V_{ν} denotes the velocity in the x direction.

There is no acceleration in the 2 direction. Incorporating a viscous resistance into the failure criterion for the sediment gives

$${}^{T}xz = (\gamma'\cos\alpha \cdot z - nz) \tan \phi' - n\frac{\partial V}{\partial z}$$
(25)

where n denotes the viscosity of the sediment.

The viscous term is negative here because, owing to the choice of axes, the velocity gradient is negative. Substituting equation (25) into (24) gives

$$\frac{\partial^2 V_{\mathbf{v}}}{\partial z^2} - \frac{1}{a} \frac{\partial V_{\mathbf{v}}}{\partial t} = -\mathbf{b} \tag{26}$$

where
$$a = \frac{g\eta}{\gamma}$$
 (27)

and
$$b = \left\{ \frac{\gamma' \sin \alpha - (\gamma' \cos \alpha - n) \tan \phi'}{\eta} \right\}$$
(28)

Equation (26) is to be solved subject to the boundary conditions

t = 0,
$$V_v = 0;$$

t > 0 $\left\{z = 0, \frac{\partial V_v}{\partial z} = 0;\right\}$
 $\left\{z = h, V_v = 0.\right\}$

where h is the depth of the slump. This problem has been considered by Curslaw and Jaeger (1959) in the context of heat conduction and the solu-

$$v_{v} = \frac{bh^{2}}{2} \sqrt{1 - \frac{z^{2}}{h^{2}} - \frac{3z}{\pi^{3}} \sum_{n=0}^{\infty} \frac{(-1)^{n}}{(2n+1)^{3}}}$$

$$\cos \frac{(2n+1)^{2}\pi^{2}t}{7!!} = -a\frac{(2n+1)^{2}\pi^{2}t}{4h^{2}}$$
(30)

Equation (30) may be expressed in terms of a dimensionless depth factor

time factor

and velocity factor

$$\frac{2V_{v}}{h^{1/2}}$$

and plotted graphically as in Figure 12 to reveal the development of the velocity profile with increasing time. The maximum velocity occurs at the surface of the flow, and plotting the velocity factor against time factor for z = 0, it is seen from Figure 13 that for a small time a linear relationship exists. More particularly

$$\frac{2V_{v}}{bh^2} = \frac{2}{sh^2}$$
 (31)

Therefore, for small time

$$V_v = \frac{E}{Y} [\gamma' \sin \alpha - (\gamma' \cos \alpha - n) \tan \phi'] t$$
(32)

and comparing equation (32) with equation (23) one finds

$$v_{\mathbf{v}} = V_{\mathbf{r}} \tag{33}$$

In the early stages of motion the maximum velocity developed in the frictional-viscous flow will be the same as that in the purely frictional flow. The average velocity will be slightly less. For larger times the viscosity will now be more significant. Viscosity data for sediments of high concentration are scarce. However, on the basis of experiments reported by Yano and Daido (1965) values of between 0.4 and 0.5 lb (force) sec per sq ft may be used in calculations for the concentration of sediments likely to exist in an accelerating slump.

The process of transformation into a turbidity current involves the onset of turbulence and the likelihood of some mixing with overlying water due to instability and wave formation at the interface. This is a difficult problem and is by no means fully resolved at present. Among the factors that would deter a slump from transforming into a turbidity current are rapid decrease of slope inclination and the dissipation of pore pressure. It is of interest, then, to adopt a relationship that has been applied to the steady state flow of a turbidity current in order to find a velocity at which it may be assumed that transformation is complete, and then, for an assumed slump, compute the time required to achieve this velocity. The degree of dissipation at this time can also be estimated.

A clump 30 ft thick is assumed to have occurred on a slope of 5 degrees and following Kuenen (1352) it is assumed that the Chezy equation is valid when the turbidity current is created. It is also assumed that the bulk density of the sediment is three times the submerged density. From the Chezy equation a velocity of 58.5 ft per sec is obtained. If it be further assumed that the angle of shearing resistance is 20 degrees and

is 0.8, this velocity is attained in only 340 seconds. It is evident that the degree of dissipation of pore pressure for a slump of this size after 340 seconds is negligible for all but the coarsest sediment. It seems probable that in the experiments carried

out by Buffington (1961) the amount of sediment was so small that, aggravated by spreading, the drainage path was sufficienty small to allow almost instantaneous dissipation of the excess pore pressure.

For a slump to turn into a turbidity current, the analysis presented here shows that it is necessary that at failure the strength be reduced sufficiently to permit the acceleration of the mass, and that deeper slumps will transform more readily because, other things being equal, the dissipation of pore pressure will be less.

CONCLUDING REMARKS

Much of this study is necessarily speculative because of the paucity of reliable strength data for submarine sediments. It is evident that a more profound understanding of submarine slumping requires this information, as well as more detailed studies of topography, occurrence of slumping, and rate of accumulation of material in varying sedimentary environments. The development of underconsolidation in deltas and submarine campon heads deserves special attention.

The transformation of a moving slump into a turbidity current is a complicated problem involving both soil and fluid mechanics. Conditions that must be satisfied for the onset of turbulence and the development of the dispersive forces that arise and maintain the sediment in suspension are not well understood. The mixing with overlying water is an important factor in the development of a turbidity current, and controls its density. This process must be clarified before the mechanics of turbidity currents of high density can be founded on a firm physical base.

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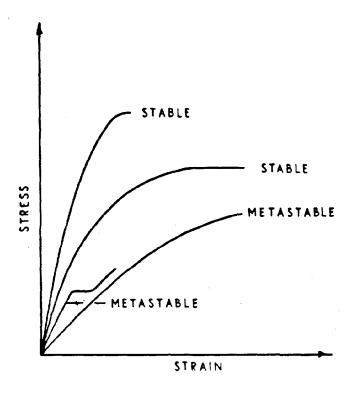
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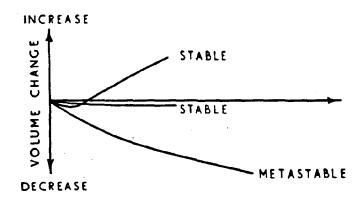


FIGURE 1. DIAGRAMMATIC STRESS — STRAIN RELATIONS FOR STABLE AND METASTABLE SEDIMENTS.

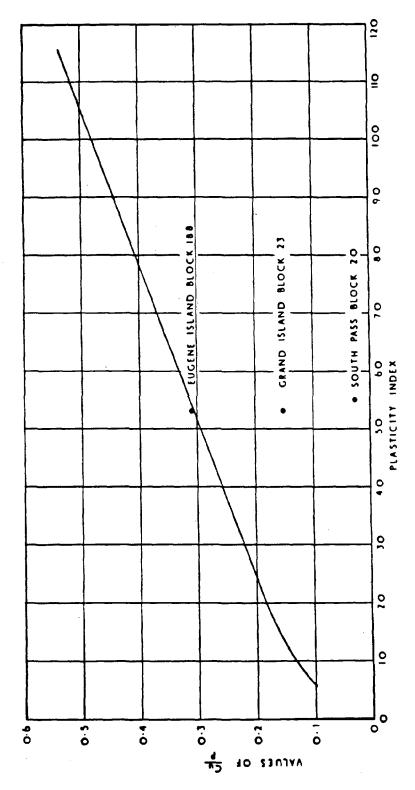


FIGURE 2. RELATION BETWEEN UNDRAINED STRENGTH AND PLASTICITY INDEX FOR NORMALLY CONSOLIDATED SEDIMENT.

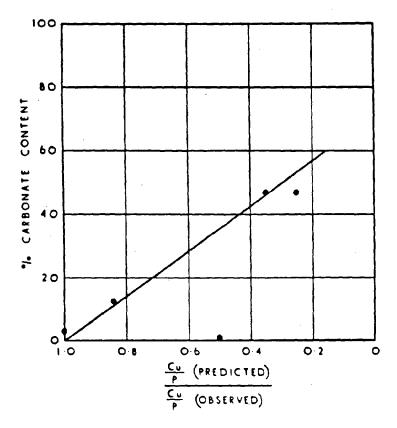


FIGURE 3. INFLUENCE OF CARBONATE CONTENT ON UNDRAINED STRENGTH
OF SEDIMENTS FROM EXPERIMENTAL MOHOLE.

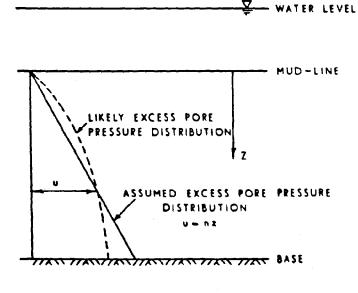


FIGURE 4. AN UNDERCONSOLIDATED STRATUM.

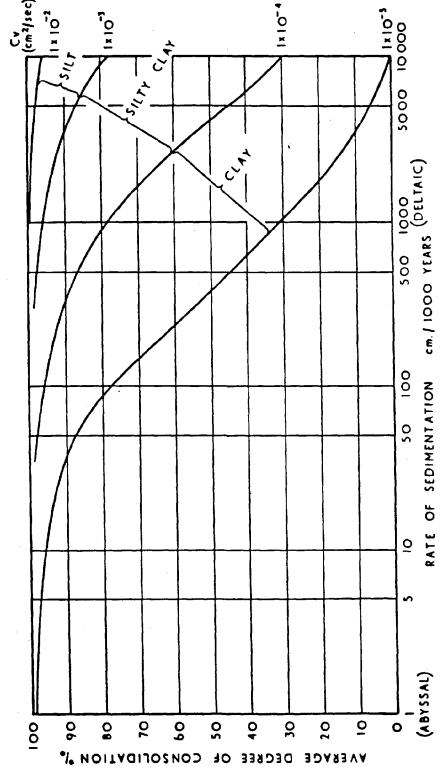


FIGURE 5. RELATION BETWEEN RATE OF SEDIMENTATION AND DEGREE OF CONSOLIDATION FOR 15 m LAYER.

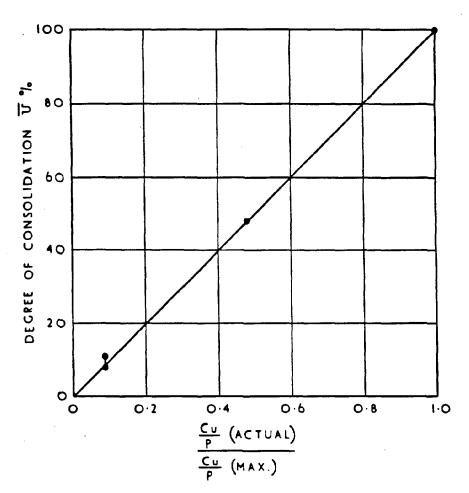


FIGURE 6. INFLUENCE OF UNDERCONSOLIDATION ON UNDRAINED STRENGTH OF MISSISSIPPI DELTA SEDIMENTS.

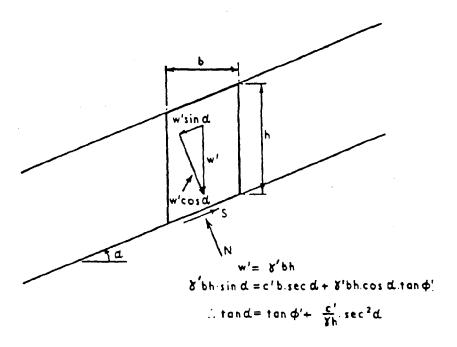


FIGURE 7. EQUILIBRIUM OF INFINITE SLOPE UNDER DRAINED CONDITIONS.

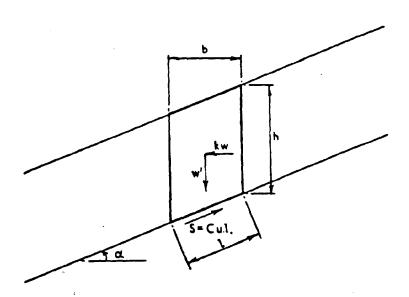


FIGURE 8. EQUILIBRIUM OF INFINITE SLOPE UNDER UNDRAINED CONDITIONS.

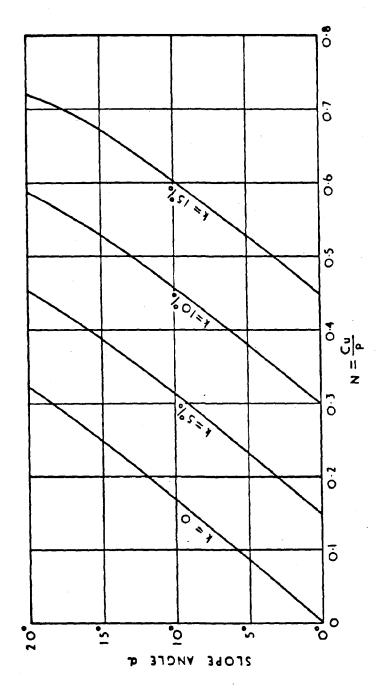
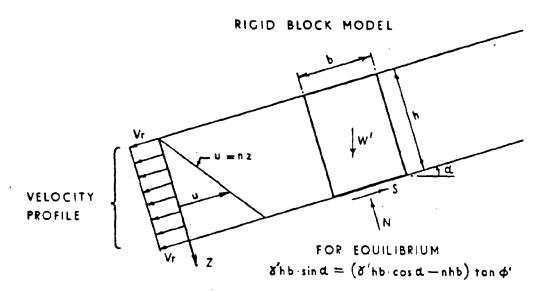


FIGURE 9. RELATION BETWEEN SLOPE ANGLE AND UNDRAINED STRENGTH FOR AN INFINITE SLOPE AT LIMITING EQUILIBRIUM AND SUBJECT TO AN EARTHQUAKE ACCELERATION K PERCENT OF GRAVITY.



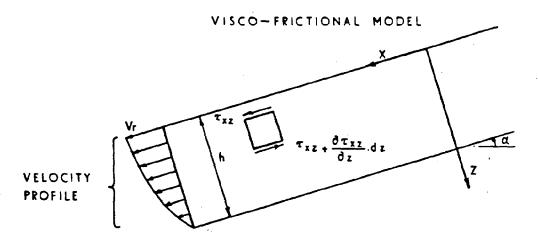


FIGURE 10. ACCELERATION OF AN INFINITE SLOPE.

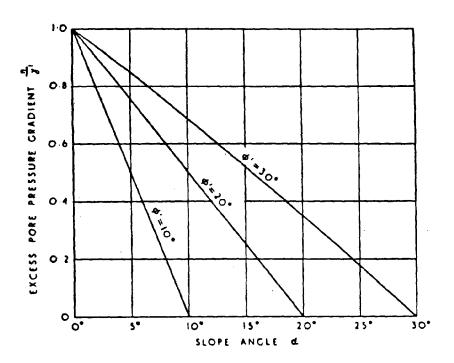


FIGURE 11. RELATION BETWEEN EXCESS PORE PRESSURE AND INCLINATION FOR AN INFINITE SLOPE AT LIMITING EQUILIBRIUM.

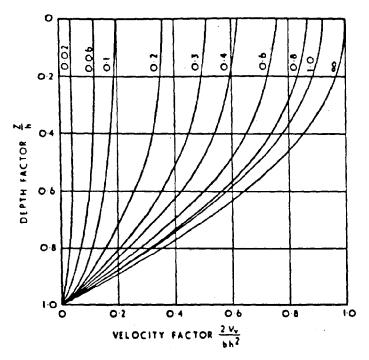


FIGURE 12. VELOCITY PROFILES FOR INCREASING VALUES OF TIME FACTOR $\frac{at}{h^2}$.

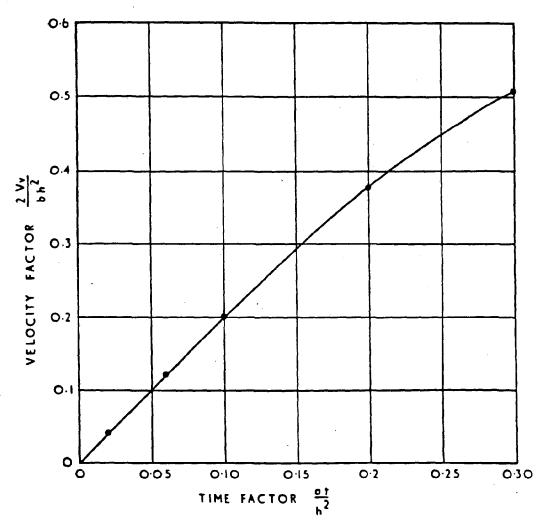


FIGURE 13. RELATION BETWEEN VELOCITY FACTOR AND TIME FACTOR AT $\frac{7}{h}$ = 0.

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